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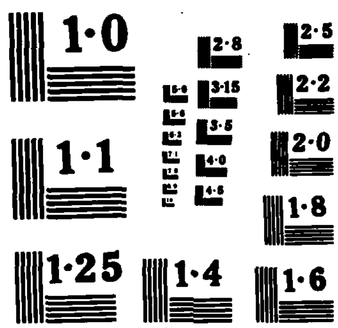

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MICROCOPY RESOLUTION TEST CHART

AD-A156 537

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MERRIMACK RIVER BASIN
TEMPLE, NEW HAMPSHIRE

SOUHEGAN RIVER WATERSHED DAM NO. 26

NH 00207

NHWRB 234.08

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1979

DISCLAIMER NOTICE

SOUHEGAN RIVER WATERSHED DAM NO. 26
NH 00207

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MERRIMACK RIVER BASIN
HILLSBOROUGH COUNTY, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

PHASE I REPORT

Identification No.:	NH 00207
NHWRB No.:	234.08
Name of Dam:	SOUHEGAN RIVER WATERSHED DAM NO. 26
Town:	Temple
County and State:	Hillsborough County, New Hampshire
Stream:	Blood Brook, a tributary of the Souhegan River
Date of Inspection:	May 1, 1979

BRIEF ASSESSMENT

The Souhegan River Watershed Dam No. 26 is located on Blood Brook, a tributary of the Souhegan River, approximately 2 miles upstream of West Wilton, New Hampshire. (Township of Temple, N.H.). The dam is an earth embankment 692 feet long and 79 feet high with a drop inlet service spillway structure and a 30 inch outlet conduit. An emergency spillway 320 feet wide is cut into the left abutment.

The dam is owned by the New Hampshire Water Resources Board. It was designed by the Soil Conservation Service for the purpose of flood protection in the Souhegan River Watershed.

The drainage area of the dam covers 4.9 square miles and is made up primarily of mountainous woodland. The dam impounds only 29.6 acre-feet at low stage but has a maximum impoundment of 1486 acre-feet. The dam is INTERMEDIATE in size and its hazard classification is HIGH since significant property damage and loss of life could result in the event of a dam failure.

The test flood for this dam is the Probable Maximum Flood. The peak inflow for this flood is 13,760 cfs. Because of storage, the resulting peak discharge is 11,900 cfs compared to a spillway capacity of 12,544 cfs. The water surface would be at elevation 928.8 feet (MSL) or 0.2 feet below the top of the dam for this flood.

The dam is in GOOD condition at the present time. Remedial measures to be undertaken by the owner include: Backfilling animal burrows, tire ruts, and drainage gully in embankment slopes; mowing embankment slopes; providing access to the riser structure; repairing outlet conduit; operating the drain gate as part of the annual inspection program; and developing a formal written emergency flood warning system.

No conditions were observed which require further investigation.

The remedial measures outlined above should be implemented within two years of receipt of this report by the owner, however, the program of annual technical inspections should be continued.

THE STATE OF NEW HAMPSHIRE
WILLIAM S. ZOINO
REGISTERED PROFESSIONAL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS ANTHONY CAMPAGNA, JR.
No. 21,006
CIVIL
STATE OF CALIFORNIA

William S. Zoino
N.H. Registration 3226

Nicholas A. Campagna, Jr.
California Registration 21006

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the Test Flood should not be interpreted as necessarily posing a highly inadequate condition. The Test Flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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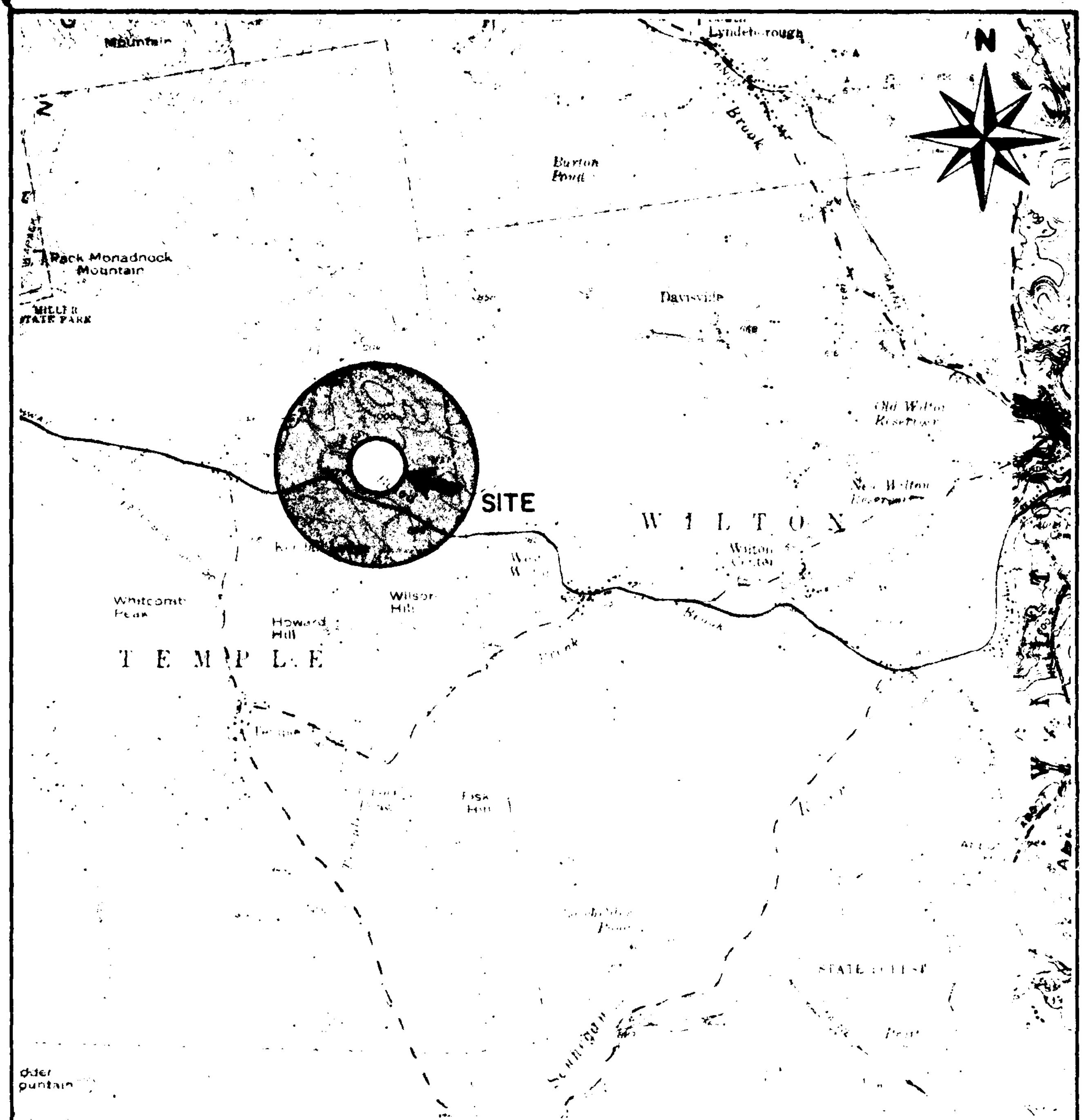
Overview from right abutment

Overview from left abutment



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FROM: USGS PETERBOROUGH & MILFORD - N.H. QUADRANGLE MAPS

FILE No. 2327

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN

SOUHEGAN RIVER WATERSHED
DAM No. 26

NEW HAMPSHIRE

SCALE AS NOTED
DATE MAY 1979

PHASE I INSPECTION REPORT

SOUHEGAN RIVER WATERSHED DAM NO. 26

SECTION 1

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to GZD under a letter of March 30, 1979 from Colonel John P. Chandler, Corps of Engineers. Contract No. DACW 33-79-C-0058 has been assigned by the Corps of Engineers for this work.

(b) Purpose

1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.

3) Update, verify, and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams, and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

The Souhegan River Watershed Dam No. 26 is located on Blood Brook approximately 2 miles upstream of West Wilton, New Hampshire. It can be reached from McRea Road which intersects Powers Road which intersects State Route 101 in Temple, New Hampshire. The dam is shown on USGS Peterborough, N.H. quadrangle at approximately coordinates N 42° 50.5', W 71° 50.0'. (See location map on page v). Figure 1 of Appendix B is a site plan for this dam.

(b) Description of Dam and Appurtenances

The dam consists of an earth embankment with an earthfill cutoff trench below the embankment, a principal spillway with a reinforced concrete riser and outlet pipe, and an emergency spillway located at the left abutment. The length of the embankment is 692 feet. There is a separate emergency spillway which is 320 feet wide at the control section.

1) Embankment (See pgs. B-3, B-4, B-5 & B-8)

The embankment is made up primarily of silty fine sand (Designation SM using the Unified Soil Classification System). It is 692 feet long and is a maximum of 79 feet high. The upstream slope is 3 horizontal to 1 vertical; the downstream slope is 2.5 horizontal to 1 vertical; and the width of the crest is 14 feet.

Beneath the embankment is an earthfill cutoff trench of variable width at the bottom. According to available plans, it is constructed of the same silty fine sand material as the embankment. The cutoff trench was designed and constructed to extend through sand and gravel layers to firm bedrock or glacial till.

There is a berm approximately 10 feet wide on the upstream slope at approximately normal pool elevation. The purpose of this berm is wave erosion protection. It is constructed of the same silty fine sand material as the embankment.

2) Principal Spillway (See pgs. B-6 & B-7)

The principal spillway consists of a reinforced concrete drop inlet structure with a sluice gate controlled inlet pipe and two uncontrolled orifice inlets and an outlet pipe supported on a concrete cradle.

The riser structure is 52 feet high and 9.5 feet wide normal to the axis of the dam. It is 5 feet long parallel to the embankment for the bottom 12 feet. It reduces to 4.5 feet for the next 32 feet and flares to 14.5 feet long at the top. The walls of the structure are 15 inches thick for the bottom 12 feet and 12 inches thick for the remaining distance. The top slab is 10 inches thick.

At the base of the structure is a transition toe approximately 10 feet long. The purpose of this structure is to reduce the channel from 5 feet by 2.5 feet at the riser structure to 30 inches in diameter at the outlet conduit. This toe has been monolithically cast with the base of the inlet structures.

At the base of the structure is a 24 inch diameter, vertical lift, sluice gate inlet which is controlled by a wheel operated bench stand with a rising stem. A 24 inch diameter, asphalt coated, corrugated metal pipe extends 40 feet upstream from the lift gate into the impoundment pool. Plans indicate a reinforced concrete inlet structure at the upstream end of this pipe which is protected by a trash rack of galvanized steel bars placed on an incline across the opening.

The "low stage inlet" is an uncontrolled opening approximately 19 feet above the sluice gate invert. It is one foot, 10 inches wide and 14 inches high and is located in the upstream face of the riser structure. The water flows over this orifice and drops into the riser structure. It is protected by a trash rack assembly approximately 9 feet high and 4 feet, 6 inches wide. This assembly is fabricated from galvanized steel angle sections.

The "high stage inlet" consists of two openings approximately 51 feet above the sluice gate invert. They are 7.5 feet wide and 15 inches high and are located in the left and right sides of the flared portion of the riser structure. They are protected by a galvanized steel grating 3 feet high placed in front of each high stage opening and 5 steel angles placed in the sloping section below each opening. A 30 inch diameter manhole permits access into the riser structure.

The riser structure is drained by a 30 inch diameter reinforced concrete pressure pipe. It is approximately 368 feet long and drops approximately 3.75 feet over that length. The pipe penetrates the downstream side of the riser structure and is supported by a 7.5 inch thick concrete cradle within the embankment. Plans indicate 9 concrete anti-seep collars cast around the pipe within the embankment.

The downstream end of the conduit and cradle extend approximately 12 feet downstream of the embankment. The cradle is supported by a reinforced concrete tee bent. The top flange of this bent is 12 inches thick, 18 inches deep, and 4.75 feet wide. The discharge conduit outlets into a stone revetted plunge pool.

3) Emergency Spillway (See pg. B-3)

The emergency spillway was excavated in the left abutment. It curves to the right around the embankment and is 320 feet wide at the control section. It is approximately 700 feet long and lies approximately 7 feet below the top of the embankment. The side slopes are 4 horizontal to 1 vertical.

4) Foundation and Embankment Drainage (See pg. B-5)

A four foot wide trench drain of clean sand and gravel extends the full width of the downstream embankment. It contains two 12 inch perforated metal pipes. One extends to 150 feet to the left of the outlet conduit, and the other extends to 100 feet to the right of the outlet conduit. These pipes discharge on either side of the conduit.

A blanket drain of clean sand and gravel extends from 65 feet downstream of the centerline to the downstream toe (see pages B-4 and B-5).

(c) Size Classification

The dam's maximum impoundment of 1486 acre feet and height of 79 feet place it in the INTERMEDIATE size category according to the Corps of Engineers' Recommended Guidelines.

The hazard potential classification for this dam is HIGH because of the significant economic losses and potential for loss of life downstream which may occur in the event of dam failure. Section 5 of this report presents more detailed discussion of the hazard potential.

(e) Ownership

The dam is owned by the New Hampshire Water Resources Board, 37 Pleasant Street, Concord, New Hampshire 03301. They can be reached by telephone at area code 603-271-3406.

(f) Operator

The operation of the dam is controlled by the New Hampshire Water Resources Board. Key officiels are as follows:

George McGee, Chairman
Vernon Knowlton, Chief Engineer
Donald Rapoza, Assistant Chief Engineer

The Board's telephone number is 603-271-3406. Alternatively, the Board can be reached through the state capital at 603-271-1110.

(g) Purpose of the Dam

The purpose of the dam is to reduce downstream flooding by providing temporary storage for the runoff from 4.9 square miles of watershed. This temporary storage is released gradually through the low and high stage inlets of the principal spillway.

(h) Design and Construction History

The dam was designed by the U.S. Department of Agriculture, Soil Conservation Service in conjunction with the New Hampshire Water Resources Board. It was completed in 1965.

(i) Normal Operating Procedure

The dam is normally self regulating. The pond drain gate is operated only as part of infrequent maintenance checks.

1.3 Pertinent Data

(a) Drainage Area

The drainage area for this dam covers 4.9 square miles. It is made up primarily of mountainous woodland with some pasture and minor development.

(b) Discharge at Damsite

1) Outlet Works

Normal discharge at the site is through the 30 inch diameter outlet pipe. In the event of severe flooding water would flow over the emergency spillway at elevation 923.0 feet (MSL). The invert of the low stage orifice is at elevation 873.5 feet (MSL). The invert of the high stage orifice is at elevation 905.0 feet (MSL).

2) Maximum Known Flood

There is no data available for the maximum known flood at this damsite.

3) Ungated Spillway Capacity at Top of Dam

The capacity of the principal spillway with the reservoir at top of dam elevation (929.0 feet MSL) is 150 cfs. The capacity of the emergency spillway is 12,394 cfs at this level.

4) Ungated Spillway Capacity at Test Flood

The capacity of the principal spillway with the reservoir at test flood elevation (928.8 feet MSL) is 150 cfs. The capacity of the emergency spillway is 11,750 cfs at this level.

5) Gated Spillway Capacity at Normal Pool

There are no gated spillways with the exception of the gated pond drain inlet which is normally closed.

6) Gated Spillway Capacity at Test Flood

As previously mentioned, there are no gated spillways.

7) Total Spillway Capacity at Test Flood

The total spillway capacity at test flood elevation (928.8 feet MSL) is 11,900 cfs.

(c) Elevation (feet above MSL)

1) Streambed at centerline of dam: 855.0

2) Maximum tailwater: Unknown

3) Upstream portal invert diversion tunnel: Not applicable

4) Normal pool: 873.5

5) Full flood control pool: 923.0

6) Spillway crest:

a) Pond drain inlet: 858.5

b) Low stage inlet: 873.5

c) High stage inlet: 905.0

d) Emergency spillway: 923.0

7) Design surcharge: 927.0

8) Top dam: 929.0

9) Test flood design surcharge: 928.8

(d) Reservoir

1) Length of maximum pool: 2,560 ± ft.

2) Length of normal pool: 800 ± ft.

3) Length of flood control pool: 2,500 ± ft.

(e) Storage (acre feet)

1) Normal pool: 29.6

2) Flood control pool: 1,149

3) Spillway crest pool:

a) Low stage inlet: 29.6

b) High stage inlet: 454

c) Emergency spillway: 1,149

4) Top of dam: 1,486

5) Test flood pool: 1,474

(f) <u>Reservoir Surface</u> (acres)

1) Normal pool: 4.6

2) Flood control pool: 52.8

3) Spillway crest pool:

a) Low stage inlet: 4.6

b) High stage inlet: 23 ±

c) Emergency spillway: 52.8

4) Test flood: 59.6

5) Top of dam: 59.7

(g) <u>Dam</u>

1) Type: Earth embankment

2) Length: 692 ft.

3) Height: 79 ft.

4) Top width: 14 ft.

5) Side slopes: Upstream: 3 to 1
Downstream: 2.5 to 1

6) Zoning: Homogeneous, semi-pervious silty fine sand. Toe drain of clean sand and gravel

7) Impervious core: None

8) Cutoff: Variable width, earthfill

9) Grout curtain: None

(h) <u>Diversion and Regulating Tunnel</u>

Not applicable

(i) Spillways

1) Type:

a) Principal spillway: Reinforced concrete Drop inlet

b) Emergency spillway: Grass covered earth channel cut in left side of reservoir

2) Length of weir:

a) Pond drain inlet: 24 inch diameter pipe

b) Low stage inlet: 1.83 ft.

c) High stage inlet: 15 ft.

d) Emergency spillway: 320 ft.

3) Crest Elevation (ft. above MSL)

a) Pond drain inlet: 858.5

b) Low stage inlet: 873.5

c) High stage inlet: 905.0

d) Emergency spillway: 923.0

4) Gates: 24 inch vertical lift sluice gate on pond drain inlet

5) Upstream channel: Reservoir

6) Downstream channel: Narrow channel through gently sloping flood plain

(j) Regulating Outlet

The only regulating outlet is a 24 inch diameter pipe controlled by a wheel operated sluice gate. The pipe invert is at elevation 858.5 feet (MSL). The purpose of this outlet is pond drainage, and it is normally closed.

SECTION 2 - ENGINEERING DATA

2.1 Design Data

Among other design data available from the Soil Conservation Service are hydrologic and hydraulic computations, structural computations, a geological report, soil laboratory test results, and embankment slope stability analysis computations. This information was used extensively in the computations presented in Section 5 and Appendix D of this report.

2.2 Construction Data

"As built" plans are available for this dam and show good agreement with the design plans and the visual inspection.

2.3 Operational Data

No operational data is available as the dam is self regulating.

2.4 Evaluation of Data

(a) Availability

Sufficient data is available to permit an evaluation of the dam when combined with findings of the visual inspection.

(b) Adequacy

There is sufficient design and construction data to permit an assessment of dam safety when combined with the visual inspection, past performance, and sound engineering judgment.

(c) Validity

Since the observations of the inspection team generally confirm the available data, a satisfactory evaluation for validity is indicated.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

(a) General

The Souhegan River Watershed Dam No. 26 is in GOOD condition at the present time.

(b) Dam

1) Earth Embankment (See photo 2)

Two small animal burrows were found in the upstream slope. Tire ruts 4 to 6 inches deep and a drainage gully 6 to 10 inches deep were found in the right upstream toe of the embankment near the abutment. There is a foot path on the right downstream abutment. The upstream slope is not protected by riprap, but it is in good condition. There is debris on the upstream slope.

The toe drains were functioning with the left toe drain discharging approximately ten gallons per minute and the right toe drain discharging approximately five gallons per minute. The discharge is clear.

2) Emergency Spillway (See photo 1)

The emergency spillway is in good condition. There are wet spots in the channel but these are caused by natural groundwater or ponded runoff.

(c) Appurtenant Structure

1) Drop Inlet Service Spillway Structure (See photos 2, 4 & 6)

This structure was observed from the embankment since access to the structure was not possible. The existing ladder is too short and no extension was available.

The structure is in good condition with no evidence of spalling, cracking, or efflorescence. The mortar rubbed surface finish has been worn away by moisture intrusion. The sluice gate bench stand is in good condition. The hand wheel has been removed from the site to prevent unauthorized use. The trash racks are in good condition but are clogged with debris. The exterior ladder is too short to allow access to the structure.

2) Pond Drain Inlet Pipe

At the time of inspection the 24 inch pond drain inlet pipe was completely submerged and could not be observed.

3) Outlet Conduit (See photos 3 & 5)

The downstream end of the outlet pipe shows cracking with associated efflorescence over 5 to 10 percent of its exposed surface. These cracks are from shrinkage and lack of quality control during construction. There is no evidence of settlement or displacement of the conduit. The tee bent is completely below ground.

(d) Reservoir Area

The shore of the reservoir is generally shallow sloping woodland. It appears stable and in good condition.

(e) Downstream Channel (See photo 5)

The downstream channel is a narrow channel passing over relatively flat flood plain. The channel appears stable and in good condition. Riprap protection of the plunge pool is in good condition.

3.2 Evaluation

The dam is generally in good condition. The outlet conduit is in fair condition. The potential problems noted during the visual inspection are listed below.

a) Drainage gully and tire ruts in right upstream abutment.

b) Animal burrows in the upstream slope.

c) Debris on upstream slope.

d) Downstream end of outlet conduit shows cracking.

e) No ladder extension available for access to the riser structure.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures

No written operational procedures are available for this dam. The dam is normally self regulating.

4.2 Maintenance of Dam

An annual inspection is made jointly by the New Hampshire Water Resources Board and the Soil Conservation Service. Recommendations resulting from this inspection are implemented by the NHWRB.

4.3 Maintenance of Operating Facilities

Operation of the sluice gate for the pond drain inlet is checked approximately once every four or five years by NHWRB.

4.4 Description of Warning System in Effect

There is no warning system in effect.

4.5 Evaluation

The established operational procedures for this dam are generally satisfactory. Additional emphasis on routine maintenance will assist the owners in assuring the long-term safety of the dam. A formal, written, downstream emergency flood warning system should be developed for this dam.

SECTION 5 - HYDROLOGY/HYDRAULICS

5.1 Evaluation of Features

(a) General

Souhegan River Watershed Dam No. 26 is a Soil Conservation Service (SCS) flood control dam on Blood Brook in Temple, New Hampshire. The dam is about two miles upstream of the village of West Wilton and 6 miles upstream of the confluence of Blood Brook and the Souhegan River. The upstream drainage area is 4.9 square miles with mountainous topography.

The dam itself is a 692 foot long earthen embankment with a grass-lined earth emergency spillway, 320 feet wide. The principal spillway consists of three orifices located on a concrete riser in the reservoir. Flow from the orifices proceeds under the dam through a reinforced concrete pipe.

(b) Design Data

The data sources available for Souhegan River Watershed Dam No. 26 include the Soil Conservation Service's (SCS) "Hydrology and Hydraulics" Design Calculations. These calculations include Storage-Elevation and Stage-Discharge curves for the dam, and the routing of storms of various magnitudes through the reservoir. These calculations are dated 1964 and 1965.

Also available for this dam are SCS "Maintenance Checklist" reports on dam inspections dated June 2, 1977 and June 15, 1978, and the Soil Conservation Service Design plans, dated 1966.

The SCS established the elevation of the low stage outlet (873.5 feet MSL) at the top of the 50-year sediment pool. The elevation of the two high stage outlets (905 feet MSL) was established at the 10-year flood stage in the reservoir. The emergency spillway crest was set at the 100-year flood stage (923 feet MSL), and the dam crest (929 feet MSL) was set slightly above the Probable Maximum Floor (PMF) stage.

(c) <u>Experience Data</u>

No records of flow or stage are known to be available for Souhegan River Watershed Dam No. 26.

(d) <u>Visual Observations</u>

The emergency spillway is a grass-lined earth channel, with its crest at 923 feet MSL and 4:1 sideslopes. Outflow from the emergency spillway does not feed into Blood Brook immediately. It runs through a minor channel and a swamp to the north before joining Blood Brook about 2,000 feet downstream. The principal spillway consists of a concrete riser structure in the reservoir with three orifices. The flow from these orifices combines in the riser and flows under the dam through a 30 inch reinforced concrete pipe 368.3 feet long.

Downstream of the dam, Blood Brook runs about 6,500 feet before reaching the first development, two houses 15 feet above the streambed and 500 feet upstream of New Hampshire Highway 101. The brook passes under Highway 101, a heavily traveled road, through a bridge with a low chord 13 feet above the streambed.

For the next 4,000 feet to the confluence of Blood and Temple Brooks, Business Route 101 parallels Blood Brook. The village of West Wilton begins slightly upstream of the confluence, and there are two small dams, two bridges and eight houses along Blood Brook in this reach.

Below the confluence with Temple Brook, Business Highway 101 continues to parallel Blood Brook, which passes 3 houses, a gift shop, and a restaurant before leaving West Wilton.

The next development, 2,000 feet downstream of West Wilton, is a house 10 to 15 feet above the streambed. Four-thousand feet downstream of that house there is an abandoned mill and mill pond, with a house 15 feet above the streambed. Highway 101 parallels Blood Brook in this area.

Below the abandoned mill pond, Blood Brook's flood plain widens somewhat and Highway 101 moves away from the brook in the 4,000 feet to the Highway 31 crossing. This bridge has a low chord less than 15 feet above the streambed. There are three houses approximately 20 feet above the stream and a junkyard 15 feet above the stream at this crossing.

Eight hundred feet downstream of the Highway 31 bridge, Blood Brook enters the Souhegan River. This confluence is about one mile upstream of a group of 30 to 40 houses 15 feet above the Souhegan's streambed, and four miles upstream of the town of Wilton, New Hampshire.

(e) Test Flood Analysis

The hydrologic conditions of interest in this Phase I investigation are those required to assess the dam's overtopping potential and its ability to safely allow an appropriately large flood to pass. This requires using the discharge and storage characteristics of the structure to evaluate the impact of an appropriately sized Test Flood. The original hydraulic and hydrologic design calculations of the SCS are available for this dam. (See page B-10).

Guidelines for establishing a recommended Test Flood based on the size and hazard classification of a dam are specified in the "Recommended Guidelines" of the Corps of Engineers. The impoundment of between 1,000 and 50,000 acre feet and the height of less than 100 feet classify this dam as an INTERMEDIATE size structure.

The appropriate hazard classification for this dam is HIGH because of the significant economic losses and potential for loss of life downstream in the event of dam failure. As shown in the Dam Failure Analysis section, the increase in flooding caused by failure would pose a threat to life and property in the village of West Wilton and at numerous other locations along Blood Brook. Other impacts of dam failure include possible damage to heavily traveled roads, to several small roads, and to several dams on Blood Brook (see Dam Failure Analysis section).

As shown in Table 3 of the Corps of Engineers' "Recommended Guidelines," the appropriate Test Flood for a dam classified as INTERMEDIATE in size with a HIGH hazard potential would be the probable maximum flood (PMF). As part of their hydraulic and hydrologic design calculations for the dam, the SCS created a "Freeboard Hydrograph" (approximately equivalent to the PMF) and routed it through the reservoir using a storage router. The peak inflow is 13,760 cfs, which is 2,800 csm on a 4.9 square mile drainage area. This compares to the 2,160 csm given on the Corps of Engineers' "Maximum Probable Peak Flow Rates" curve assuming mountainous topography.

The SCS peak inflow of 13,760 cfs is more conservative and is therefore selected as the test flood for this dam. The SCS storage routing results in a peak outflow of 11,900 cfs, with the water surface at 928.8 feet MSL, 0.2 feet below the dam crest and 55.3 feet above normal pool. This analysis assumes a starting water surface elevation of 892.0 (MSL). This is the six day drawdown from the emergency spillway crest. The drawdown time from the emergency spillway crest to the normal pool is 9 days.

(f) Dam Failure Analysis

The peak outflow that would result from the failure of Souhegan River Watershed Dam No. 26 is estimated using the procedure suggested in the Corps of Engineers New England Division's April 1978 "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs," as clarified in a December 7, 1978 meeting at the Corps' Waltham office. Normally this procedure is carried out with dam failure assumed to occur when the water surface reaches the top of the dam. In this case, however, the outflow of 12,700 cfs with the water surface at the top of the dam (929 feet MSL) is greater than the Probable Maximum Flood (PMF) routed outflow at the dam. Also, this outflow would create serious flooding downstream prior to dam failure. As a result, dam failure would cause only a small incremental increase to flood damage in this situation. Failure is therefore assumed to occur with the water surface at the SCS Design High Water of 927 feet MSL, 2.0 feet below the top of the dam.

The discharge to Blood Brook just prior to failure is given by the Stage-Discharge curve developed in Appendix D as 6,550 cfs. The tailwater elevation prior to failure at this discharge is estimated to be 860 feet MSL.

For an assumed breach width equal to 40 percent of the dam width at the half-height, the gap in the embankment due to failure would be 141 feet. The resulting increase in flow would be 129,700 cfs or a total flow of about 136,300 cfs.

The first damage center impacted by dam failure flow would be two houses at a dirt road crossing of Blood Brook 6500 feet downstream of the dam and 500 feet upstream of New Hampshire Highway 101. The houses are about 15 feet above the streambed. The pre-failure outflow of 6,550 cfs would create 13 feet of flow in the stream. The attenuated failure flow of 57,100 cfs would increase the stage to 24 feet. This would cause extreme flooding and present

a major threat of loss of life. The New Hampshire Highway 101 bridge 500 feet downstream of the houses, which has a low chord about 13 feet above the stream, would be severely overtopped and probably destroyed by the dam failure outflow.

Blood Brook proceeds about 4,000 feet to its confluence with Temple Brook in West Wilton. The brook is parallelled by New Hampshire Business Highway 101, and passes two small dams, two bridges and 8 houses in this reach. Two of the houses are 8 to 10 feet above the streambed, three 10 to 15 feet up, and three 15 to 20 feet up. New Hampshire Business Highway 101 is about 13 feet above the streambed.

The pre-failure outflow of 6,550 cfs would create a stage of 12 feet, causing flooding in the two lowest houses and possibly the next group of three. The attenuated peak dam failure flow of 47,300 cfs would create a stage of 22 feet, causing extreme flooding and threatening loss of life at all the houses in this reach. This flow would also severely damage or destroy the two dams and two bridges in the reach, and create extreme flooding along Business Highway 101.

After its confluence with Temple Brook, Blood Brook runs 2,000 feet to the end of the town of Wilton. Development in this reach includes three houses 10 to 15 feet above the streambed, a gift shop and restaurant 12 feet up, and New Hampshire Highway 101, about 14 feet up.

The pre-failure outflow of 7,500 cfs (including an assumed inflow of 950 cfs from Temple Brook) would create a stage of 13 feet, which would cause slight flooding. The attenuated peak dam failure outflow of 43,500 cfs would create a stage of 22 feet, again causing extreme flooding and threatening loss of life.

The next damage center is a house 2,000 feet downstream of West Wilton and 10 to 15 feet above the streambed. The attenuated peak dam failure outflow of 40,000 cfs would increase the stage from 10 feet to 16 feet at this location, causing flooding at the house. New Hampshire Highway 101 would also be flooded in this reach.

The next damage center is a house 15 feet above the streambed near an abandoned mill and mill pond 4,000 feet further downstream. The attenuated peak dam failure flow of 34,100 cfs would increase the stage from 12 to 19 feet, causing flooding at the house. New Hampshire Highway 101 would also be flooded in this reach.

The next damage center is at the Highway 31 bridge across Blood Brook 4,000 feet downstream. At this location, there are three houses 20 feet above the streambed and a junkyard 15 feet above the streambed. The attenuated peak dam failure outflow of 28,000 cfs would increase the stage from 9 feet to 16 feet, which would probably cause little or no damage at this location.

About 800 feet downstream of the Highway 31 bridge, Blood Brook enters the Souhegan River. The peak dam failure flow of about 28,000 cfs would be attenuated rapidly in the larger river channel. However, there is one area where, depending on antecedent flows in the Souhegan, the dam failure outflow from Souhegan River Watershed Dam No. 26 could cause serious flooding. About 4000 feet downstream of the confluence of Blood Brook and the Souhegan River, there is a group of 30 to 40 houses about 15 feet above the bed of the river. These houses might experience flooding due to dam failure flows. About 4 miles downstream of its confluence with Blood Brook, the Souhegan enters the town of Wilton, New Hampshire.

The following chart summarizes the downstream impacts of the failure of Souhegan River Watershed Dam No. 26.

DOWNSTREAM IMPACTS OF DAM FAILURE

Location (Map, p. D-13)	Location	Number of Dwellings	Level Above Streambed (ft.)	Flow and Stage: Before Failure	Flow and Stage: After Failure	Comments
-	At dam	-	-	6,550 cfs	136,300 cfs	
1	Houses, Rt. 101 bridge	2	15±	6,550 cfs 12 ft.	57,100 cfs 24 ft.	Rt. 101 bridge severely over-topped
2	West Wilton, up-stream of Temple Brook	2 3 3	8–10 10–15 15–20	6,550 cfs 12 ft.	47,500 cfs 22 ft.	Danger of loss of life. 2 dams, 2 bridges, Rt. 101 also flooded
2	West Wilton, downstream of Temple Brook	1 restaurant 1 gift shop 3 houses	12 12 10–15	7,500 cfs 13 ft.	40,000 cfs 22 ft.	Danger of loss of life. Also floods Rt. 101
	House, 2,000 ft. down-stream of West Wilton	1	10–15	7,500 cfs 10 ft.	40,000 cfs 16 ft.	Rt. 101 flooded
	House at aban-doned mill	1	15	7,500 cfs 12 ft.	34,100 cfs 19 ft.	Rt. 101, old mill and mill dam flooded
3	Highway 31 Bridge	1 junkyard 3 houses	15 20	7,500 cfs 9 ft.	28,000 cfs 16 ft.	Probably no dam-age
4	Souhegan conflu-ence	-	-	7,500 cfs	28,000 cfs	
	Downstream on Souhegan	30–40	15±	-	-	4,000 feet down-stream. Might be flooded
	Wilton	10–15 near river	varies	-	-	Flow probably attenuated

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

There has been no significant displacement or distress which would warrant the preparation of structural stability calculations.

(b) Design and Construction Data

1) Embankment

Analysis carried out during the design and construction phase included an embankment slope stability analysis by the swedish circle method. Based on this analysis a 3 to 1 upstream slope and a 2.5 to 1 downstream slope were utilized.

2) Appurtenant Structures

A review of the structural calculations for the design of the drop inlet service spillway structure and the outlet conduit (principal spillway) revealed that these structures have been designed on the basis of sound engineering practice.

(c) Operating Records

There are no known operating records for this dam.

(d) Post Construction Changes

There have been no known construction changes since the dam was completed in 1965.

(e) Seismic Stability

The dam is located in seismic zone No. 2 and, in accordance with the recommended Phase I guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The dam and its appurtenances are generally in good condition at the present time with the exception of the outlet conduit which is in fair condition.

(b) Adequacy of Information

There is sufficient design and construction data to permit an assessment of dam safety when combined with the visual inspection, past performance, and sound engineering judgment.

(c) Urgency

The recommendations and remedial measures described herein should be implemented by the owner within two years of receipt of this phase I Inspection Report.

(d) Need for Additional Investigations

None

7.2 Recommendations

No conditions were observed which warrant further investigation.

7.3 Remedial Measures

It is recommended that the owner institute the following remedial measures:

1) Check the operability of the pond drain inlet gate as part of the annual inspection procedure.

2) Develop a downstream emergency flood warning system.

3) Maintain the program of annual technical inspections.

4) Repair downstream end of outlet conduit. This can be accomplished by removing the soil overburden back to the first pipe joint and completely encasing the pipe in concrete as far as the end of the pipe cradle. The cantilevered portion of the conduit may be maintained as the need arises.

5) Provide a means of access to the riser structure during periods of normal flow by ladder extension or suitable alternative. This need not be kept at the site, but it should be available for inspection of the riser.

6) Implement and intensify a program of diligent and periodic maintenance including, but not limited to: Mowing embankment slopes; backfilling drainage gullies, tire ruts, and animal burrows with suitable, well tamped soil; and clearing debris from trash racks.

7.4 Alternatives

There are no meaningful alternatives to the above recommendations.

APPENDIX A

VISUAL INSPECTION CHECKLIST

INSPECTION TEAM ORGANIZATION

Date: May 1, 1979

Project: NH 00207
SOUHEGAN RIVER WATERSHED DAM NO. 26
Temple, New Hampshire
NHWRB 234.08

Weather: Partly cloudy, 65 degrees

INSPECTION TEAM

Nicholas A. Campagna	Goldberg, Zoino, Dunnicliff & Associates (GZD)	Team Captain
William S. Zoino	GZD	Soils
M. Daniel Gordon	GZD	Soils
Jeffrey M. Hardin	GZD	Soils
Paul Razgha	Andrew Christo Engineers (ACE)	Structures
Carl Razgha	ACE	Structures
Tom Gooch	Resource Analysis, Inc. (RAI)	Hydrology
Robert Fitzgerald	RAI	Hydrology

Owner's Representative Present

Gary Kerr - New Hampshire Water Resources Board

SOUHEGAN RIVER WATERSHED DAM NO. 26
Temple, New Hampshire

May 1, 1979
NH 00207

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
DAM EMBANKMENT		
Crest elevation	NAL	929.0'
Current pool elevation		873.5'
Maximum impoundment to date		No data
Surface cracks		None
Pavement condition		Not applicable
Movement or settlement of crest		None
Lateral movement		None
Vertical alignment		Good
Horizontal alignment		Good
Condition at abutment and at concrete structures		Drainage gully 6-10" deep and tire ruts 4-6" deep in right upstream abutment. Foot path on right downstream abutment. Concrete outlet pipe cracking
Indications of movement of structural items on slopes		None
Trespassing on slopes		Two rodent holes 3-4" diameter on upstream slope - debris on upstream slope
Sloughing or erosion of slopes of abutments		None
Rock slope protection - riprap failures		No riprap - upstream slope in good condition
Unusual movement or cracking at or near toes	NAL	None

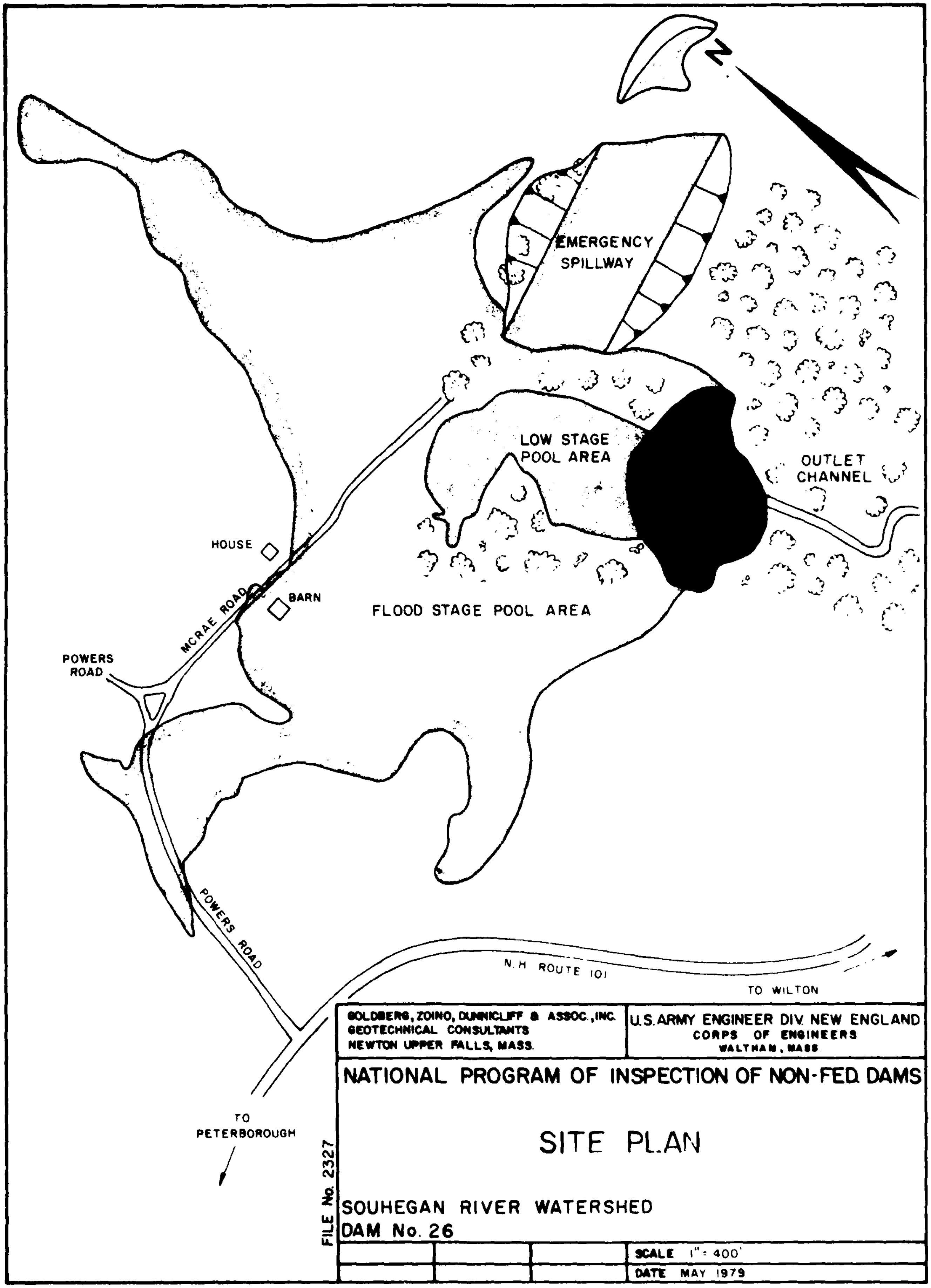
CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
<u>DAM EMBANKMENT - cont.</u>		
Unusual embankment or downstream seepage	NRC	None
Piping or boils		None
Foundation drainage features		Two toe drains functioning as below
Toe drains		Left toe drain discharging 10 to 20 gpm Right toe drain discharging 5 to 10 gpm
Instrumentation system	NRC	None
<u>PURTENANT STRUCTURES</u>		
A. Drop Inlet Service Spillway	[illegible]	
Structure		
Condition of concrete		Good
Spalling		Top surface of structure repaired over 1 sq. ft. surface area. No other deficiencies noted
Erosion		Mortar rubbed surface eroded
Cracking		None noted
Rusting or staining of concrete		None noted
Visible reinforcing		None noted
Efflorescence		None noted
Trash Racks		
Upper stage trash racks	SR	No deficiencies noted

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Trash Racks		
Lower stage trash rack	FR	Submerged, clogged with debris
Gate bench stand		No deficiencies noted
Exterior aluminum ladder		Existing ladder not accessible during normal or low flows. Ladder in good condition.
B. Reservoir Discharge Conduit		Submerged, could not be observed
C. Outlet Conduit (primary spillway)		
Condition of pipe	FR	Cracking over 5 to 10% of exposed surface area, with associated efflorescence

APPENDIX B



GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

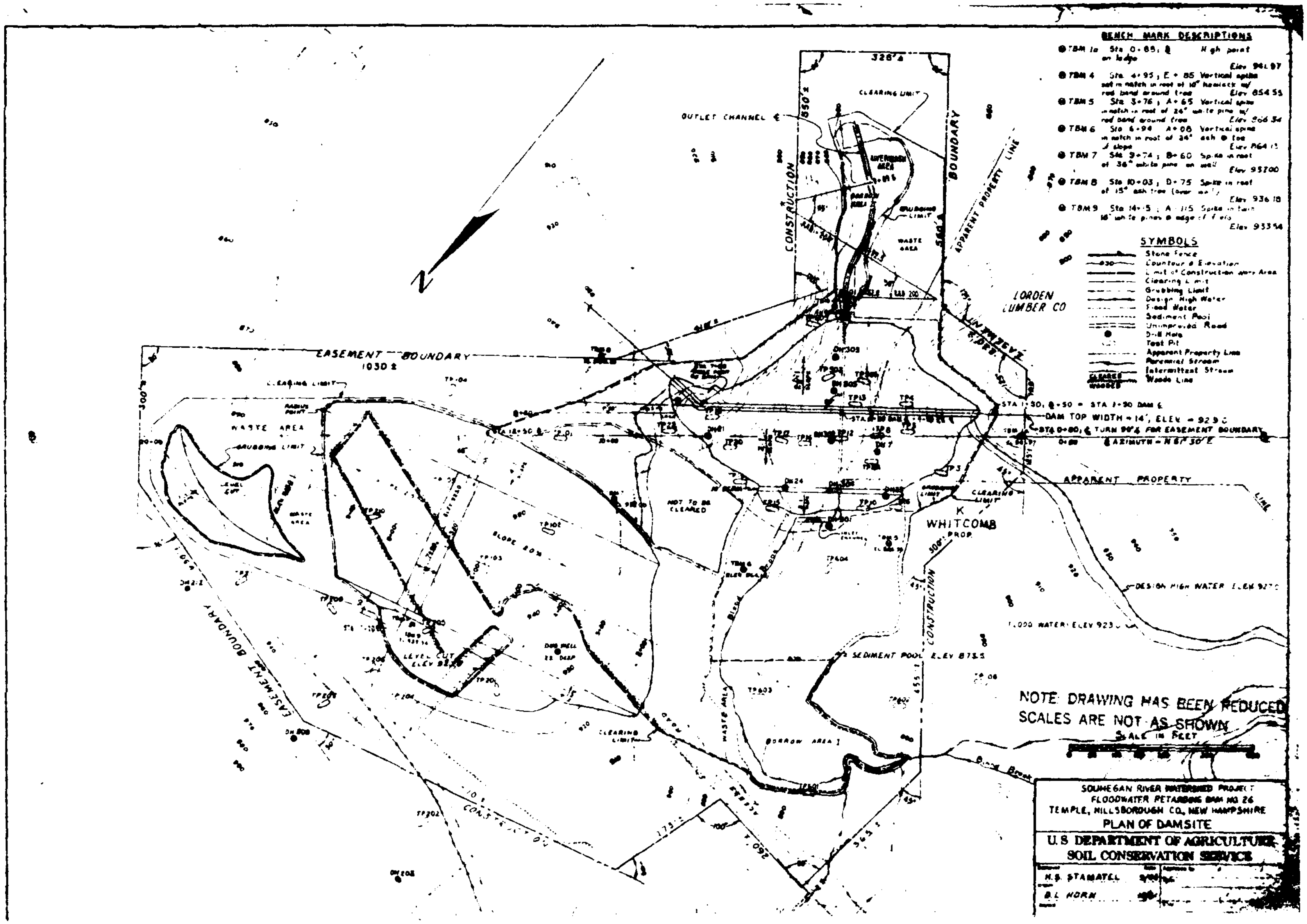
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

SITE PLAN

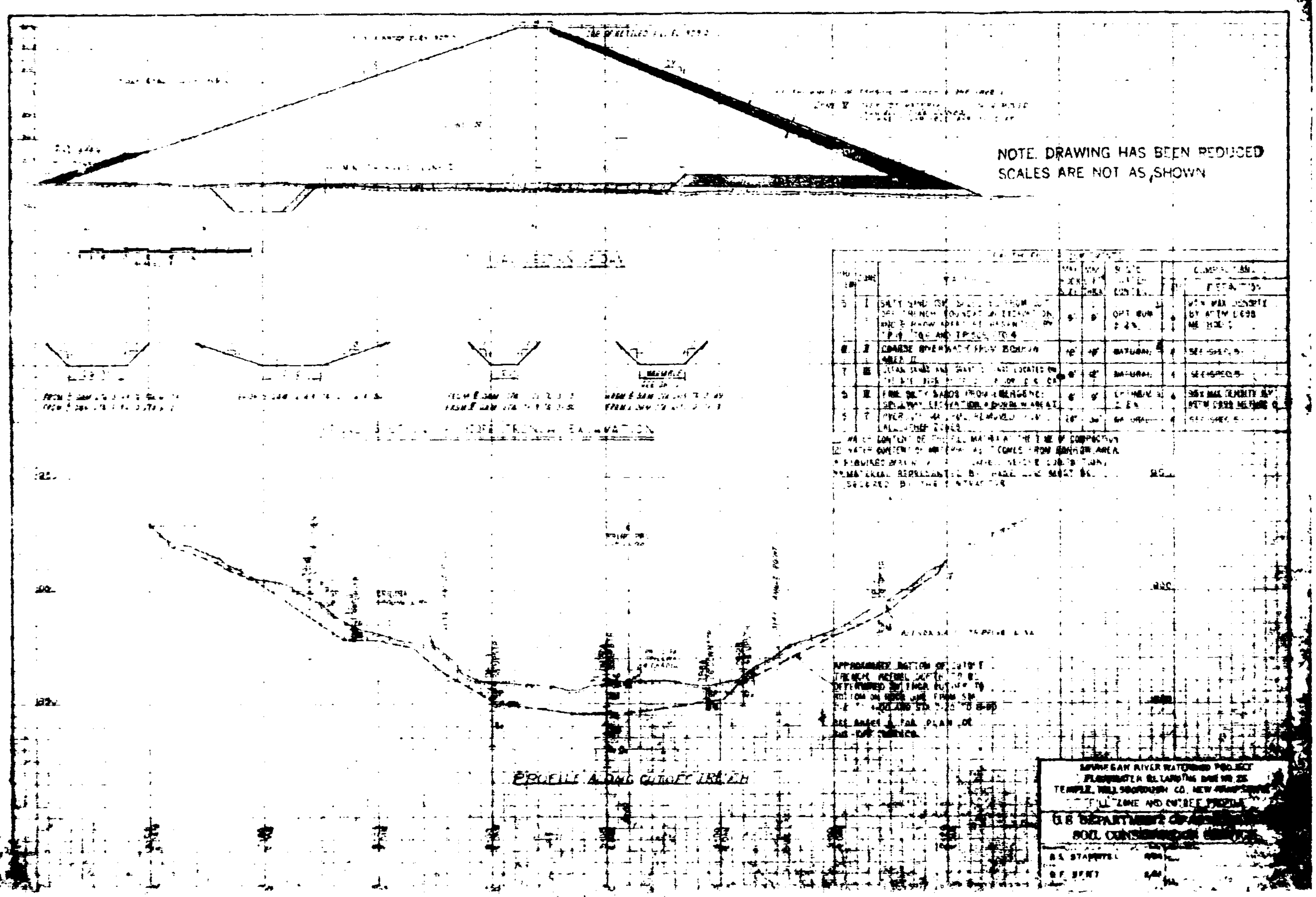
SOUHEGAN RIVER WATERSHED
DAM No. 26

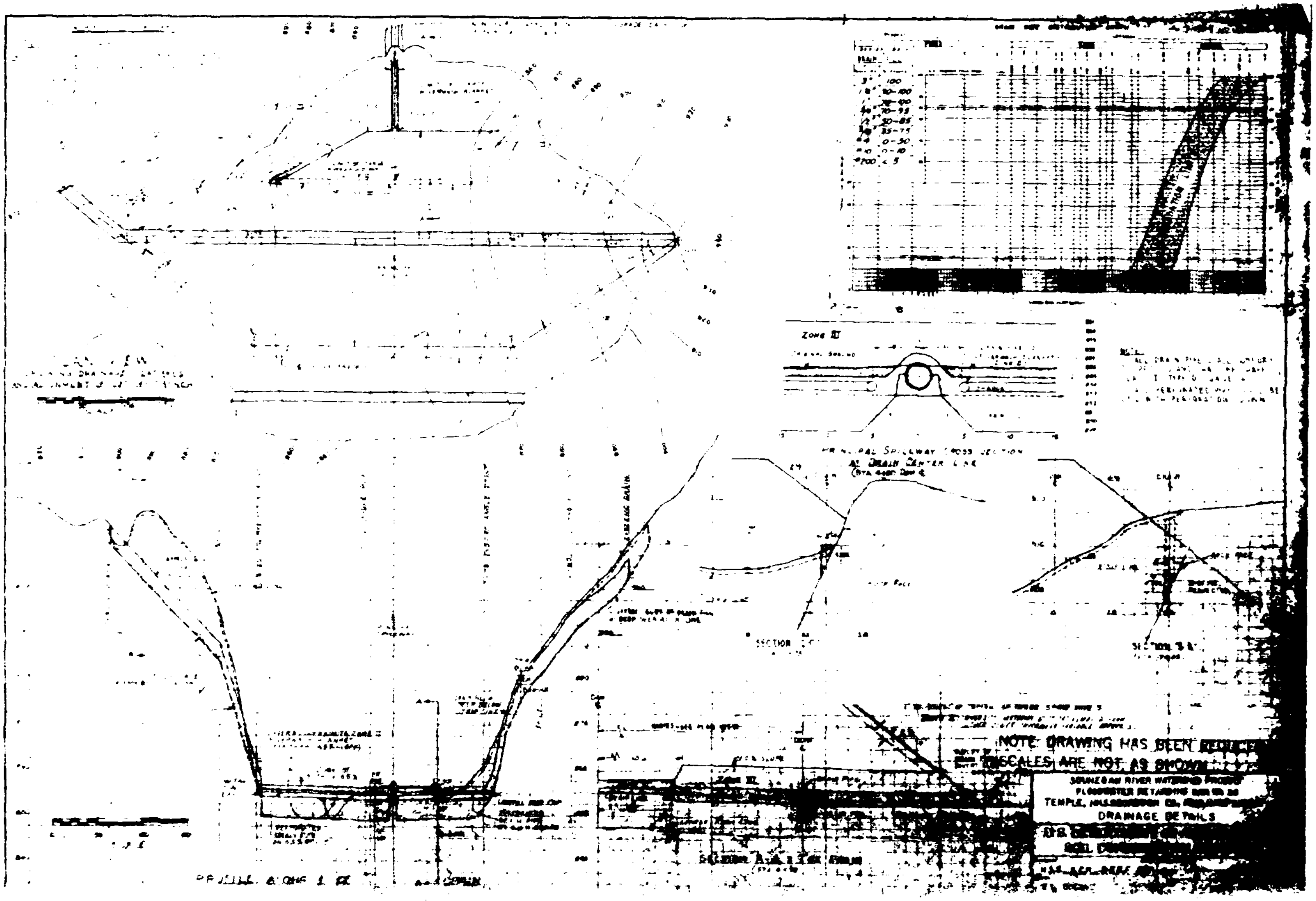
FILE No. 2327

SCALE 1" = 400'
DATE MAY 1979

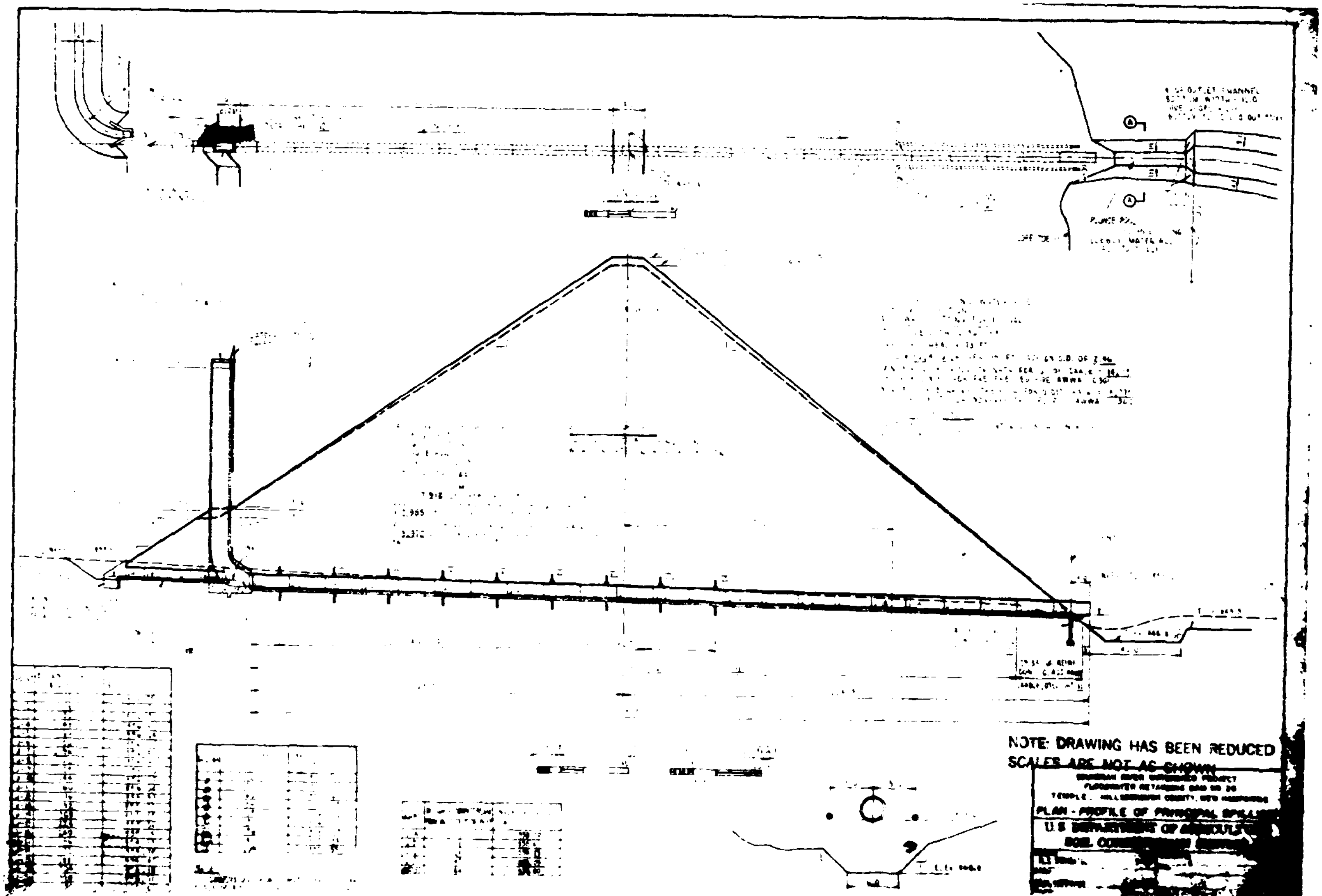


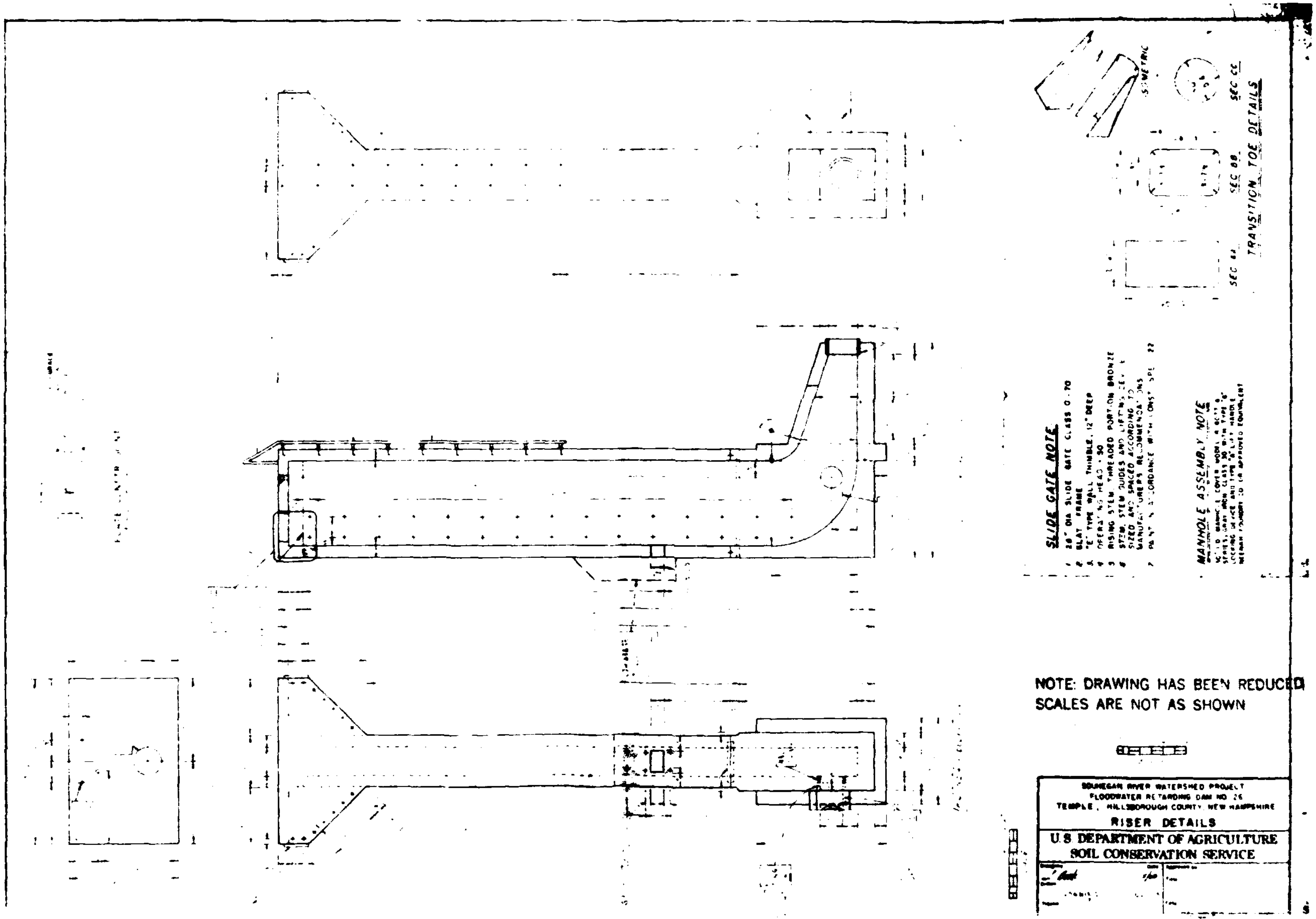
BENCH MARK DESCRIPTIONS
TBM 1a Sta 0+85; ℄ High point on ledge Elev 941.97
TBM 4 Sta 4+95; E+85 Vertical spike set in notch in root of 10" hemlock w/ red band around tree Elev 854.55
TBM 5 Sta 3+76; A+65 Vertical spike in notch in root of 24" white pine w/ red band around tree
TBM 6 Sta 6+94 A+08 Vertical spike in notch in root of 24" ash @ toe of slope
TBM 7 Sta 9+74; B+60 Spike in root of 36" white pine on wall Elev 957.00
TBM 8 Sta 10+03; D+75 Spike in root of 15" ash tree (over wall) Elev 936.18
TBM 9 Sta 14+5; A 115 Spike in twin 18" white pines @ edge of field Elev 933.54
SYMBOLS
Stone Fence
Countour & Elevation
Limit of Construction Work Area
Clearing Limit
Grubbing Limit
Design High Water
Flood Water
Sediment Pool
Unimproved Road
Drill Hole
Test Pit
Apparent Property Line
Perennial Stream
Intermittent Stream
Woods Line
CLEARED
WOODED
STA 1+50, ℄+50 = STA 1+50 DAM ℄
DAM TOP WIDTH = 14', ELEV = 929.0
STA 0+80, ℄ TURN 90° FOR EASEMENT BOUNDARY
℄ AZIMUTH = N 61° 30' E
OUTLET CHANNEL ℄
CLEARING LIMIT
CONSTRUCTION
BOUNDARY
APPARENT PROPERTY LINE
WASTE AREA
LORDEN LUMBER CO
EASEMENT BOUNDARY
1030 ±
CLEARING LIMIT
WASTE AREA
GRUBBING LIMIT
NOT TO BE CLEARED
SLOPE 20%
LEVEL CUT ELEV 873.5
APPARENT PROPERTY LINE
K WHITCOMB PROP.
CLEARING LIMIT
DESIGN HIGH WATER ELEV 927.0
FLOOD WATER ELEV 923.0
SEDIMENT POOL ELEV 873.5
BORROW AREA I
WASTE AREA
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
SCALE IN FEET
SOUHEGAN RIVER WATERSHED PROJECT
FLOODWATER RETARDING DAM NO 26
TEMPLE, HILLSBOROUGH CO., NEW HAMPSHIRE
PLAN OF DAMSITE
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
H.S. STAMATEL
B.L. HORN


NOTE. DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
PROFILE ALONG CUTOFF TRENCH
RIVER WATERSHED PROJECT
TEMPLE, CO.
ZONE AND
SOIL CONSERVATION

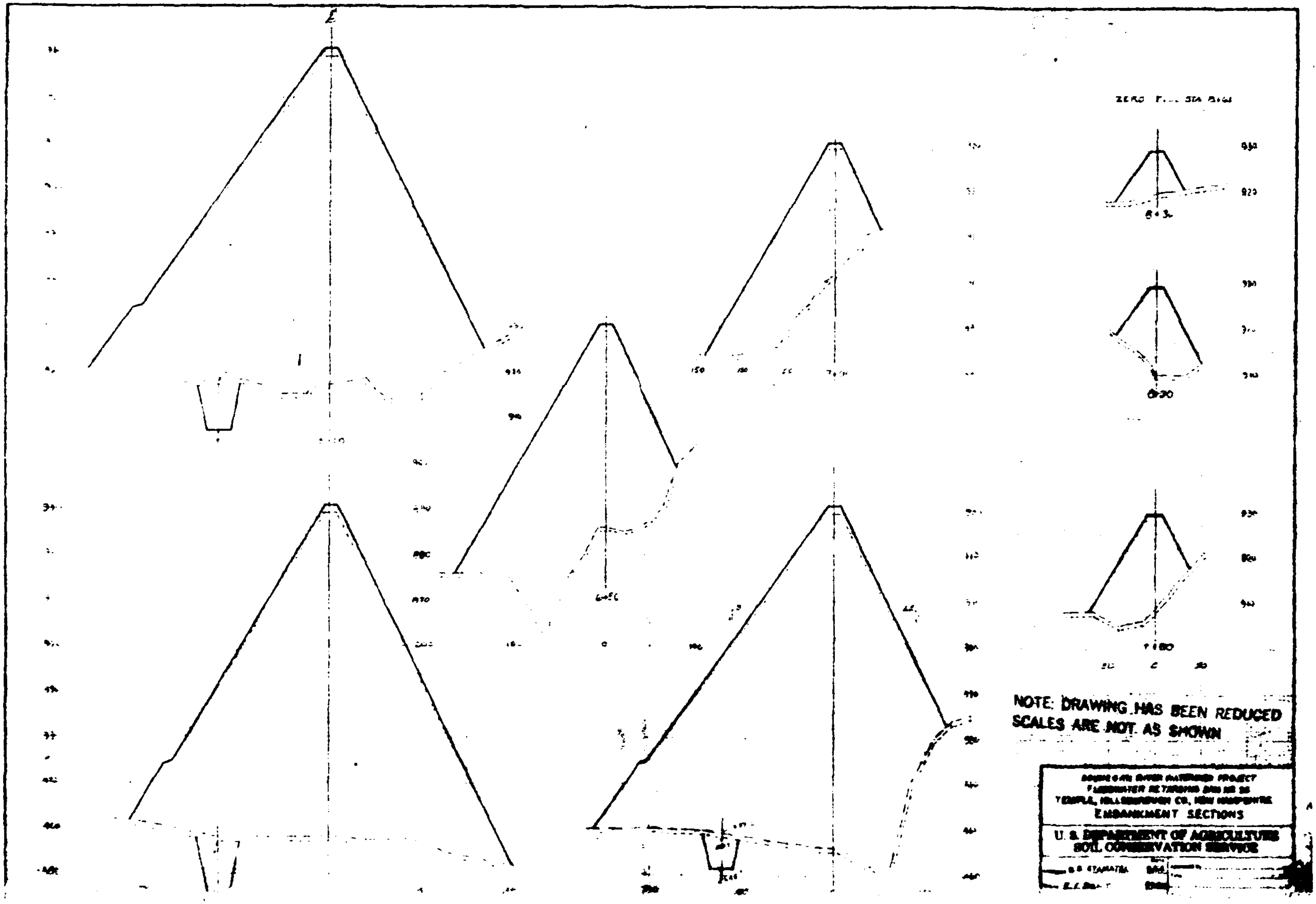


ZONE III
PRINCIPAL SPILLWAY CROSS SECTION
AT DRAIN CENTER LINE
SECTION
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
SOUHEGAN RIVER WATERSHED PROJECT
TEMPLE, HILLSBOROUGH CO.
DRAINAGE DETAILS
PROFILE ALONG ℄ OF


NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN



TRANSITION TOE DETAILS
SEC AA
SEC BB
SEC CC
ISOMETRIC
SLIDE GATE NOTE
MANHOLE ASSEMBLY NOTE
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
SOUHEGAN RIVER WATERSHED PROJECT
FLOODWATER RETARDING DAM NO. 26
TEMPLE, HILLSBOROUGH COUNTY, NEW HAMPSHIRE
RISER DETAILS
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE


ZERO FILL STA
NOTE: DRAWING HAS BEEN REDUCED
SCALES ARE NOT AS SHOWN
EMBANKMENT SECTIONS
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Poorly graded gravels.

Poorly graded sands.

Clays of high plasticity; fat clays.

(GP) FIELD CLASSIFICATION BY VISUAL INSPECTION

GP UNIFIED CLASSIFICATION BY LABORATORY

SOUHEGAN RIVER WATERSHED PROJECT

FLOODWATER RETARDING DAM NO. 28

TEMPLE, HILLSBOROUGH CO., N.H.

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

NOTE: DRAWING HAS BEEN REDUCED

SCALES ARE NOT AS SHOWN

The U.S.D.A. Soil Conservation Service (SCS) located in Durham, New Hampshire, maintains a file for this dam. Included in this file are:

1) SCS "Design Report" dated May 1965.

2) SCS "Hydrology and Hydraulics" design calculations dated 1965.

3) SCS structural design calculations dated 1965.

4) SCS "Detailed Geological Investigation of Dam Sites" dated 1963.

5) SCS soil mechanics laboratory data sheets dated 1964.

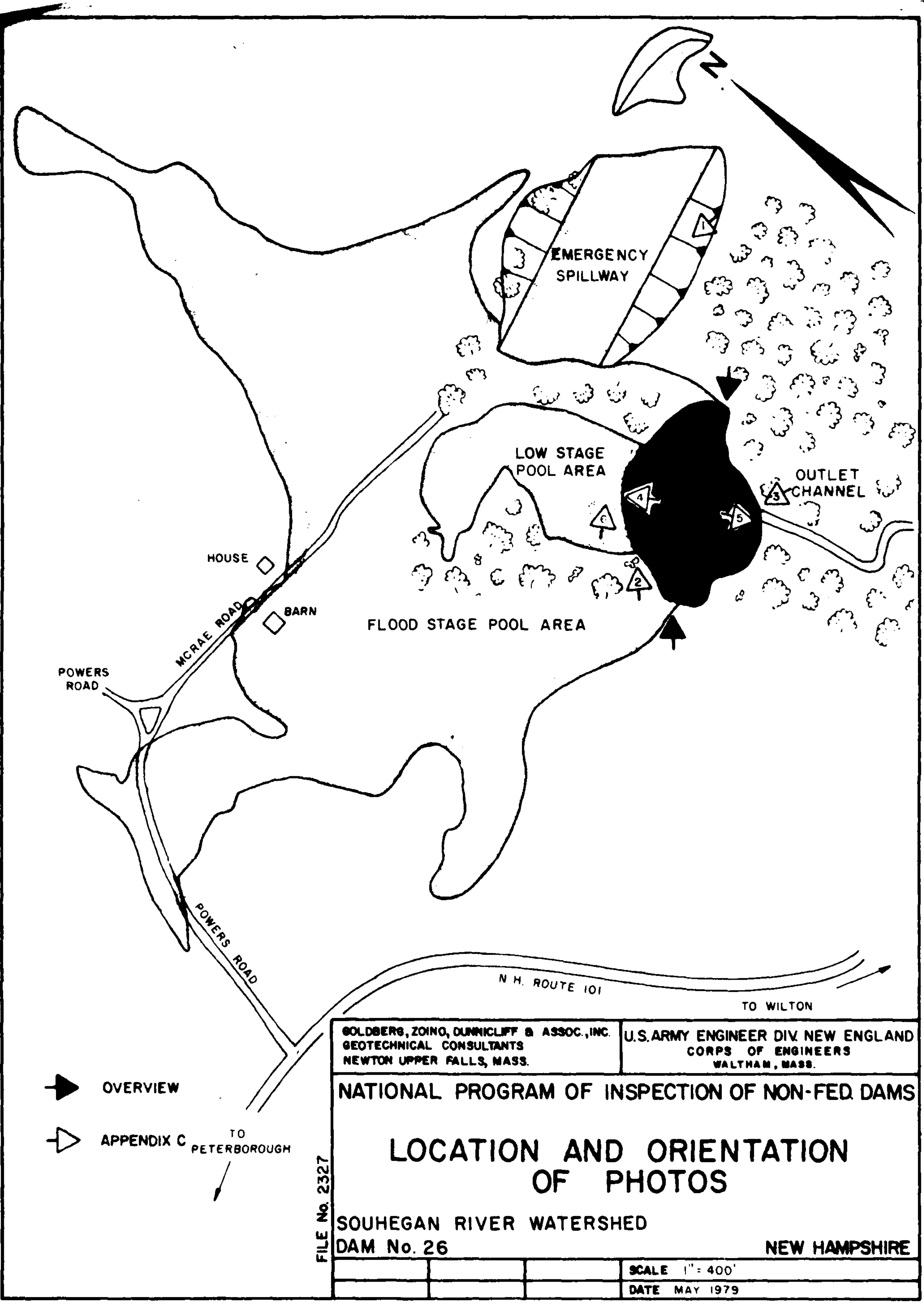
6) SCS "As Built" drawings dated 1967.

The New Hampshire Water Resources Board (NHWRB) maintains a correspondence file on this dam. Included in this file are:

1) Maintenance inspection checklists dated June 2, 1977 and June 15, 1978.

APPENDIX C

PHOTOGRAPHS



GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

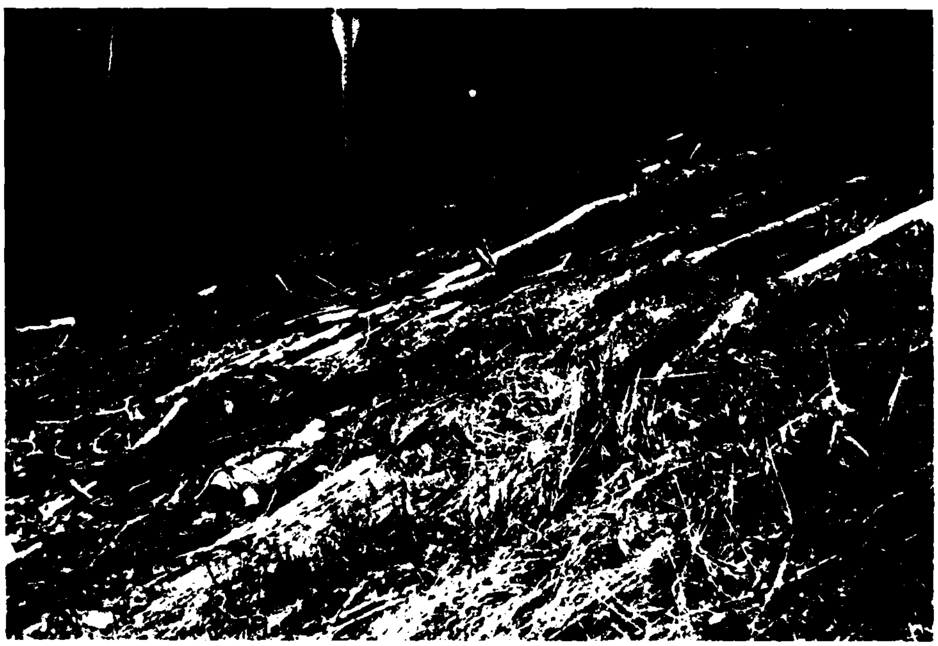
LOCATION AND ORIENTATION OF PHOTOS

SOUHEGAN RIVER WATERSHED
DAM No. 26 NEW HAMPSHIRE

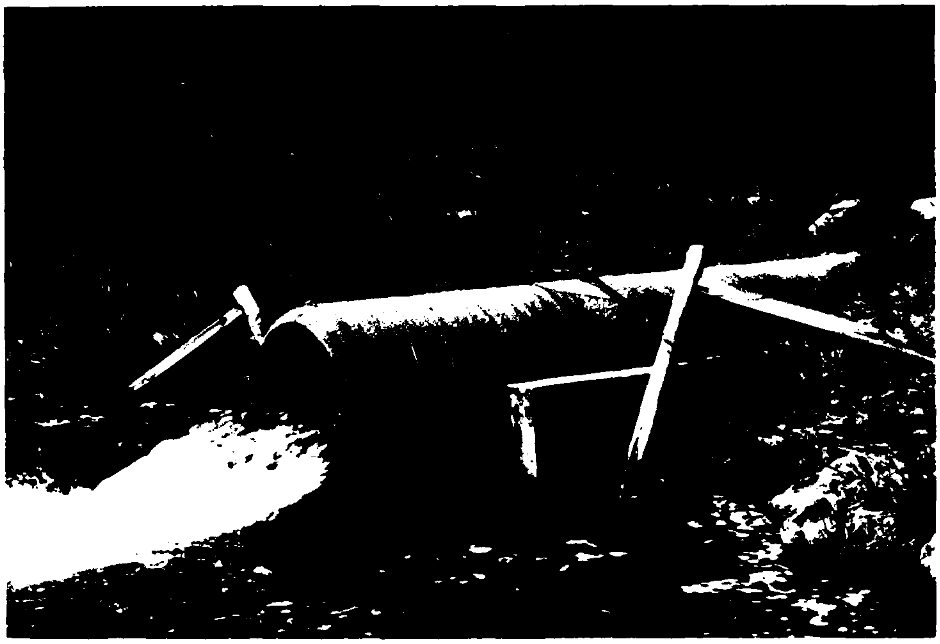
FILE No. 2327

SCALE 1" = 400'
DATE MAY 1979

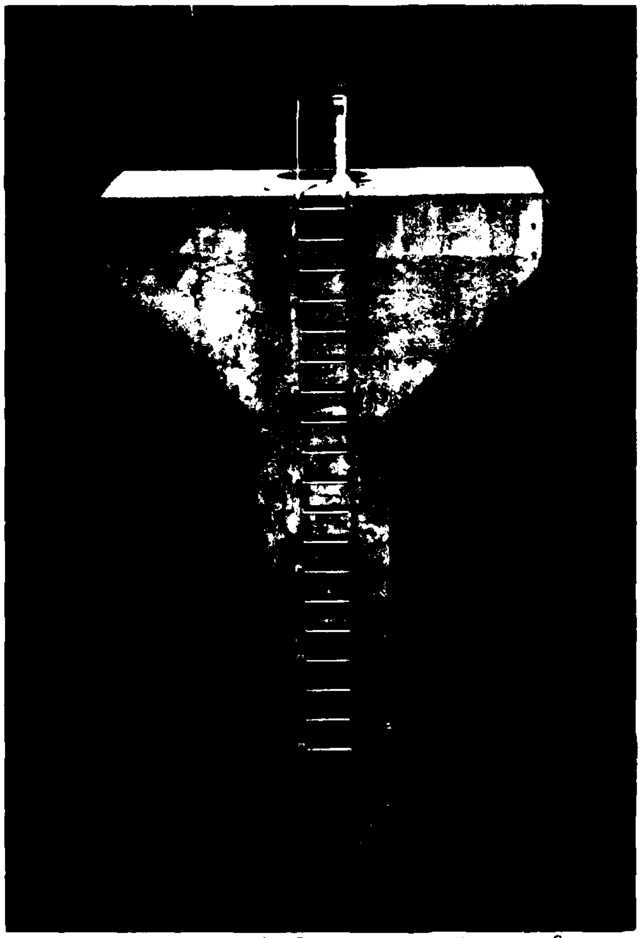
1. View of emergency spillway channel looking upstream to pond



2. View of upstream slope showing drop inlet structure and debris on slope

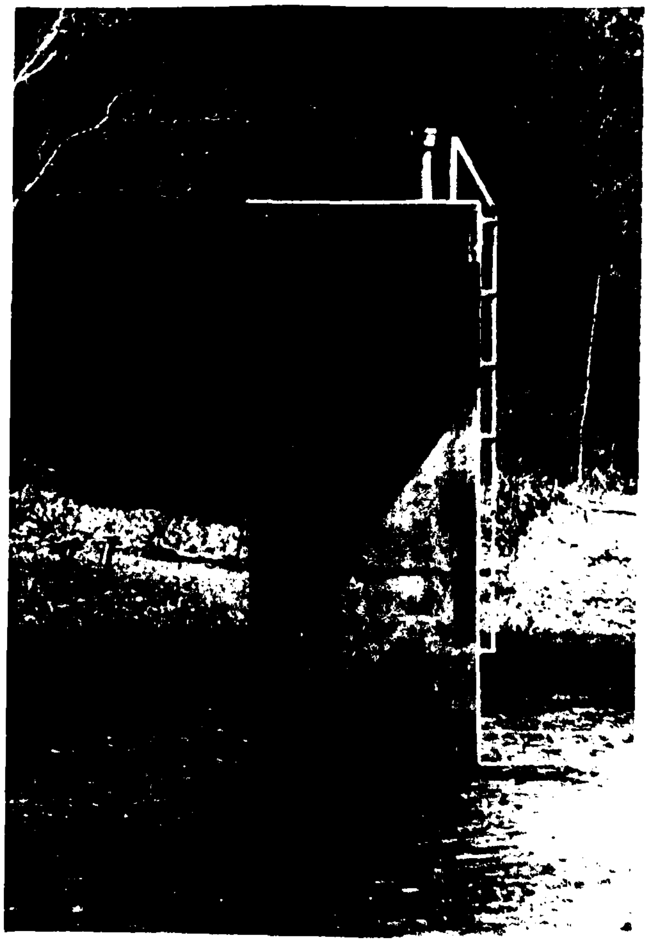


3. View of outlet pipe showing efflorescence and left toe drain outlet



4. View of drop inlet structure from embankment

5. View of downstream channel and plunge pool

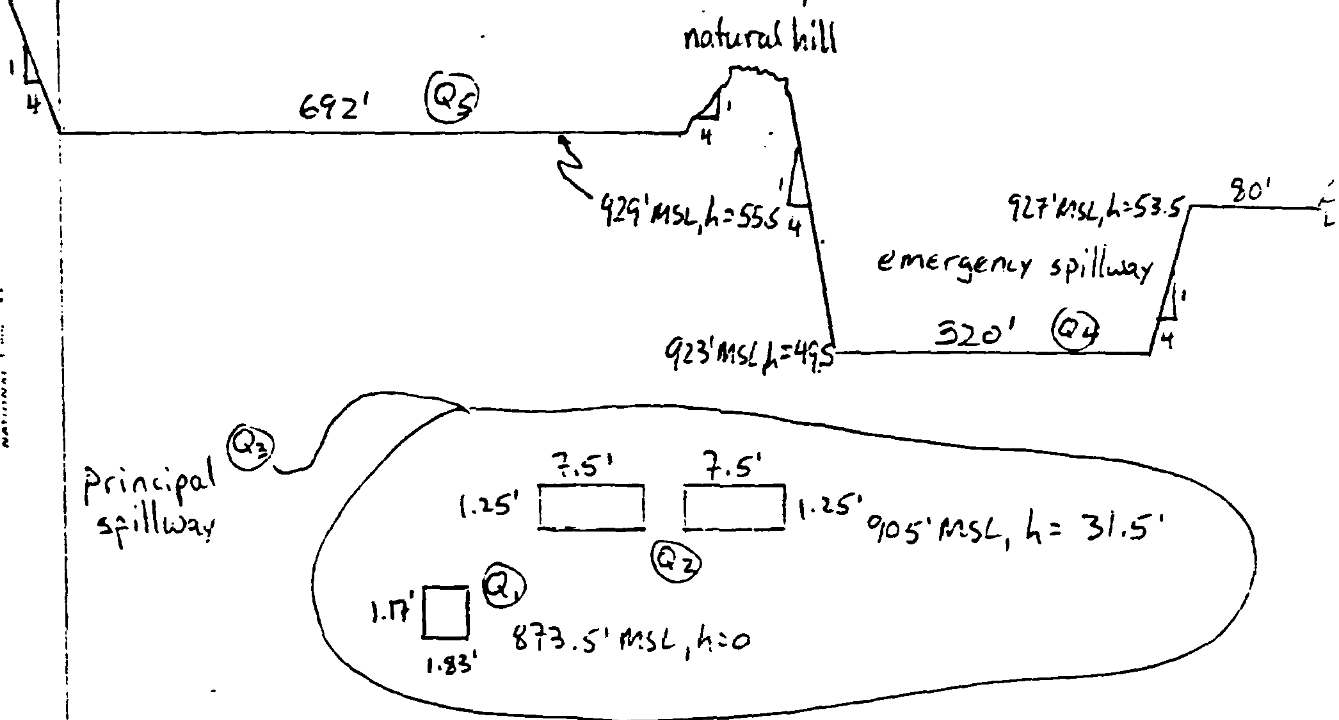


6. View of drop inlet structure showing trash racks

APPENDIX D

HYDROLOGIC/HYDRAULIC COMPUTATIONS

The information used to establish this elevation of Souhegan River Watershed Dam #26 was determined from field notes and S.C.S. construction plans dated 1965.



The 1.17' x1.83' orifice and the two 1.25' x 7.5' orifices are on a riser structure in the Reservoir. The flows from these outlets combine in the riser and flow under the dam through a 30" r.c. pipe with its upstream invert at 856.75' MSL and its downstream invert at 853' MSL. The pipe is 368.3' long. At high flows the pipe controls total outflow from the riser, which is called "principal spillway" outflow.

There is also a "pond drain inlet" which flows into the riser. The inlet is 44 ft. of 24" r.c.p. with its invert at 858.5' MSL. This inlet is not generally open, and will be assumed to be closed for these calculations.

Stage - Discharge Computations

(Q1) for $0 < h < 1.17'$

$$Q_1 = 3.3\ (1.83)\ h^{3/2}$$

C = 3.3 for sharp-crested conc. weir

for $h > 1.17'$

$$Q_1 = .65(2.14)\sqrt{2g}\ (h-.58)^{1/2}$$

$$= 11.16\ (h-.58)^{1/2}$$

C = .65 for orifice (sheet 19, SCS calcs)

(Q2) for $h < 31.5'$

$$Q_2 = 0$$

for $31.5' < h < 32.75'$

$$Q_2 = 3.3(15)\ (h-31.5)^{3/2}$$

for $h > 32.75'$

$$Q_2 = .65(1.25)(15)\ (8.02)\sqrt{h-32.1}$$

$$= 97.7\ (h-32.1)^{1/2}$$

(Q3) Q_3 is the smaller value of

$Q_3 = Q_1 + Q_2 \longrightarrow$ orifice control

or $Q_3 = 12.67\ (h+17.25)^{1/2}$ * $\longrightarrow$ pipe control

(Q4) Q_4 is given on p. 21 of SCS "Hydrologic and Hydraulic Design Calculations". The table on the next page gives S.C.S. figures:

* pp. 17-18 of SCS design calcs.

h (ft. above low flow outlet)	elevation (ft MSL)	Q_4 (cfs)
49.5	923	0
49.7	923.2	64
49.9	923.4	160
50.5	924	560
51.5	925	1792
53.5	927	6144
55.5	929	12,544

Q_5 for $h < 55.5$

$$Q_5 = 0$$

for $h > 55.5$

$$Q_5 = 2.6\,(692)(h-55.5)^{3/2} + 2\,(4)(h-55.5)\,(.5(h-55.5))^{3/2}$$

The BASIC program on pp. 4-8 gives the Stage-Discharge Relationship for this dam.

```
LIST
100 REM - STAGE/DISCHARGE CURVE FOR SOUHEGAN RIV. WATERSHED DAM # 26
110 REM - STORED ON TAPE B-1 FILE 4
120 PAGE
130 REM - THE D1 ARRAY CONTAINS EMERGENCY SPILLWAY OUTFLOW DATA
140 DIM D1(2,7)
150 DATA 49.5,49.7,49.9,50.5,51.5,53.5,55.5
160 DATA 0,64,160,560,1792,6144,12544
170 READ D1
180 PRINT USING 190:
190 IMAGE 10T"DISCHARGE FOR SOUHEGAN RIVER WATERSHED DAM NUMBER 26"
200 PRINT USING 210:
210 IMAGE 10T"   AS A FUNCTION OF HEAD ABOVE THE LOW FLOW OUTLET"
220 PRINT USING 230:
230 IMAGE // 2T"HEAD"3X"ELEVATION"30T"DISCHARGE"
240 PRINT USING 250:
250 IMAGE 1T"(FEET)"2X"(FT. MSL)"32T"(CFS)"
260 PRINT USING 270:
270 IMAGE 22T "TOTAL      PRINCIPAL      EMERGENCY      TOP OF DAM"
280 PRINT USING 290:
290 IMAGE 22T "           SPILLWAY       SPILLWAY"
300 FOR H=0 TO 56
310 Q1=0
320 Q2=0
330 Q3=0
340 Q4=0
350 Q5=0
360 Q1=3.3*1.83*H↑1.5
370 IF H<1.17 THEN 550
380 Q1=11.16*(H-0.58)↑0.5
390 IF H<31.5 THEN 550
400 Q2=3.3*15*(H-31.5)↑1.5
410 IF H<32.75 THEN 550
420 Q2=97.7*(H-32.1)↑0.5
430 REM - THE EMERGENCY SPILLWAY FLOW (Q4) IS DETERMINED BY LINEAR
```

D.4

P.5

```
440 REM -   INTERPOLATION OF THE VALUES IN ARRAY D1.
450 IF H<49.5 THEN 550
460 IF H<55.5 THEN 500
470 REM - LINEAR EXTRAPOLATION BEYOND D1 CURVE
480 Q4=D1(2,7)+(H-D1(1,7))/(D1(1,7)-D1(1,6))*(D1(2,7)-D1(2,6))
490 GO TO 550
500 FOR I=1 TO 7
510 IF D1(1,I)>H THEN 530
520 NEXT I
530 Q4=(D1(2,I)-D1(2,I-1))*((H-D1(1,I-1))/(D1(1,I)-D1(1,I-1)))
540 Q4=Q4+D1(2,I-1)
550 Q6=17.67*(H+17.25)↑0.5
560 Q3=Q1+Q2
570 REM - THE LOWER VALUE OF Q6 VS. Q1+Q2 IS CONTROLLING.
580 IF Q3<Q6 THEN 620
590 Q3=Q6
600 IF H<55.5 THEN 620
610 Q5=2.6*692*(H-55.5)↑1.5+2*4*(H-55.5)*((H-55.5)*0.5)↑1.5
620 T1=Q3+Q4+Q5
630 E=H+873.5
640 PRINT USING 650:H,E,T1,Q3,Q4,Q5
650 IMAGE 1T,3D.1D,8D.1D,11D,12D,15D,14D
660 NEXT H
670 END
```

DISCHARGE FOR SOUHEGAN RIVER WATERSHED DAM NUMBER 26
AS A FUNCTION OF HEAD ABOVE THE LOW FLOW OUTLET

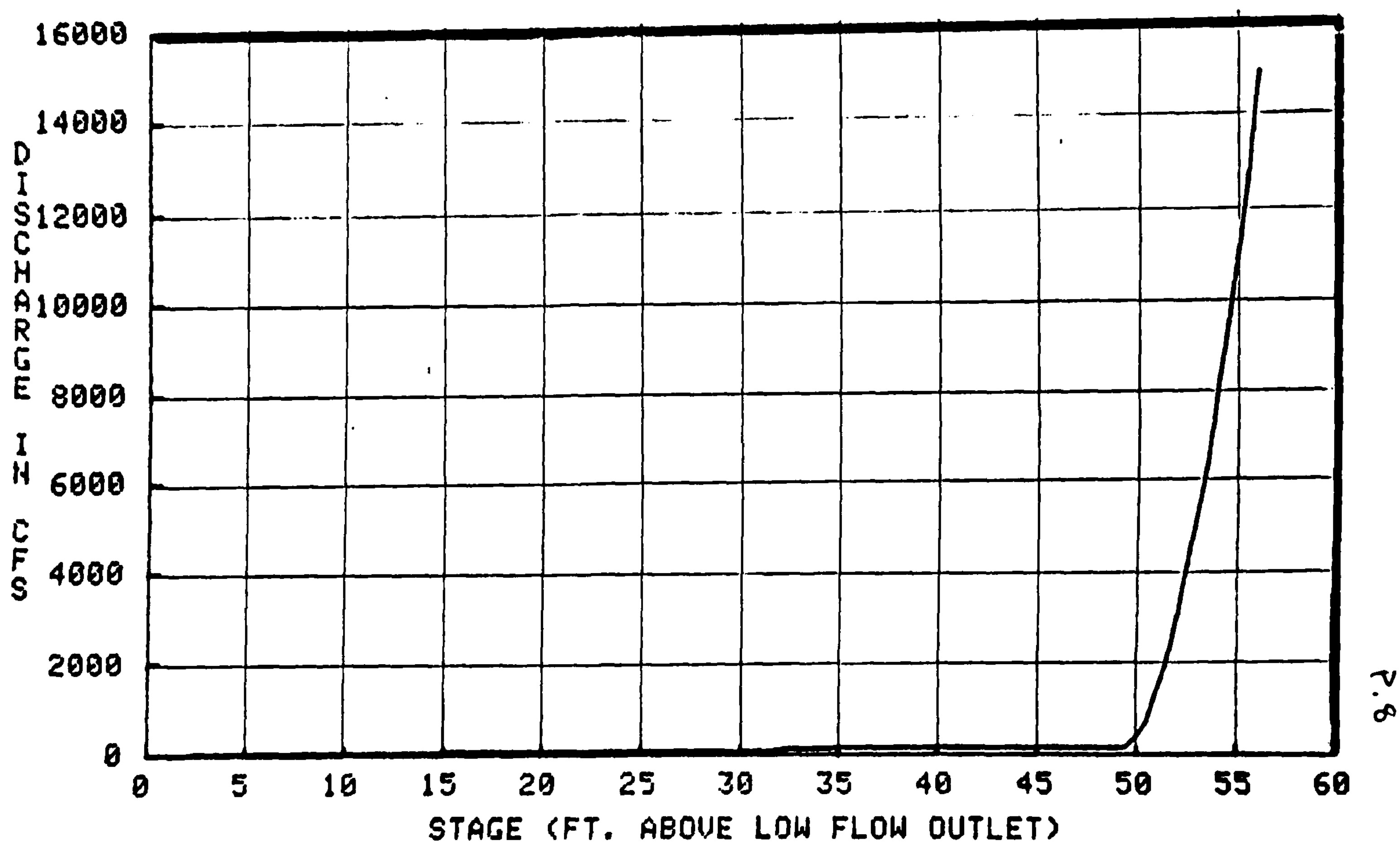
P.6

HEAD (FEET)	ELEVATION (FT. MSL)	TOTAL	DISCHARGE (CFS) PRINCIPAL SPILLWAY	EMERGENCY SPILLWAY	TOP OF DAM
0.0	873.5	0	0	0	0
1.0	874.5	6	6	0	0
2.0	875.5	13	13	0	0
3.0	876.5	17	17	0	0
4.0	877.5	21	21	0	0
5.0	878.5	23	23	0	0
6.0	879.5	26	26	0	0
7.0	880.5	28	28	0	0
8.0	881.5	30	30	0	0
9.0	882.5	32	32	0	0
10.0	883.5	34	34	0	0
11.0	884.5	36	36	0	0
12.0	885.5	38	38	0	0
13.0	886.5	39	39	0	0
14.0	887.5	41	41	0	0
15.0	888.5	42	42	0	0
16.0	889.5	44	44	0	0
17.0	890.5	45	45	0	0
18.0	891.5	47	47	0	0
19.0	892.5	48	48	0	0
20.0	893.5	49	49	0	0
21.0	894.5	50	50	0	0
22.0	895.5	52	52	0	0
23.0	896.5	53	53	0	0
24.0	897.5	54	54	0	0
25.0	898.5	55	55	0	0
26.0	899.5	56	56	0	0

P.7

27.0	900.5	57	57	0	0
28.0	901.5	58	58	0	0
29.0	902.5	59	59	0	0
30.0	903.5	61	61	0	0
31.0	904.5	62	62	0	0
32.0	905.5	80	80	0	0
33.0	906.5	125	125	0	0
34.0	907.5	126	126	0	0
35.0	908.5	128	128	0	0
36.0	909.5	129	129	0	0
37.0	910.5	130	130	0	0
38.0	911.5	131	131	0	0
39.0	912.5	133	133	0	0
40.0	913.5	134	134	0	0
41.0	914.5	135	135	0	0
42.0	915.5	136	136	0	0
43.0	916.5	137	137	0	0
44.0	917.5	138	138	0	0
45.0	918.5	139	139	0	0
46.0	919.5	141	141	0	0
47.0	920.5	142	142	0	0
48.0	921.5	143	143	0	0
49.0	922.5	144	144	0	0
50.0	923.5	372	145	227	0
51.0	924.5	1322	146	1176	0
52.0	925.5	3027	147	2880	0
53.0	926.5	5204	148	5056	0
54.0	927.5	7893	149	7744	0
55.0	928.5	11094	150	10944	0
56.0	929.5	14932	151	14144	637

STAGE-DISCHARGE CURVE FOR SOUHEGAN R. W. DAM # 26



Storage - Elevation Curve

The following Storage - Elevation Curve was taken from SCS Hydrology and Hydraulics calculations, pp. 6-9, dated 1/31/64.

elevation (ft MSL)	Stage (h) (Ft. above low flow outlet)	Current Storage (Ac - Ft.)	Available storage (after 50 yrs.) (Ac - Ft).
858		0	0
860		.2	0
865		4.2	0
870		15.2	0
873.5	0	29.6	0
875	1.5	35.8	5.5
880	6.5	67.8	35.2
885	11.5	113	77.6
890	16.5	172	135
895	21.5	247	210
900	26.5	339	301
905	31.5	454	417
910	36.5	599	562
915	41.5	778	740
920	46.5	994	956
925	51.5	1253	1216
[illegible]	56.5	149[illegible]	1459

The Storage-Elevation Curve is given on p. 11

For the drainage area of 3150 acres, 1 inch of runoff = $1/12\,(3150) = 262.5$ ac-ft.

$$1\text{ Ac-Ft} = \frac{1}{262.5} = .00381\text{'' of runoff}$$

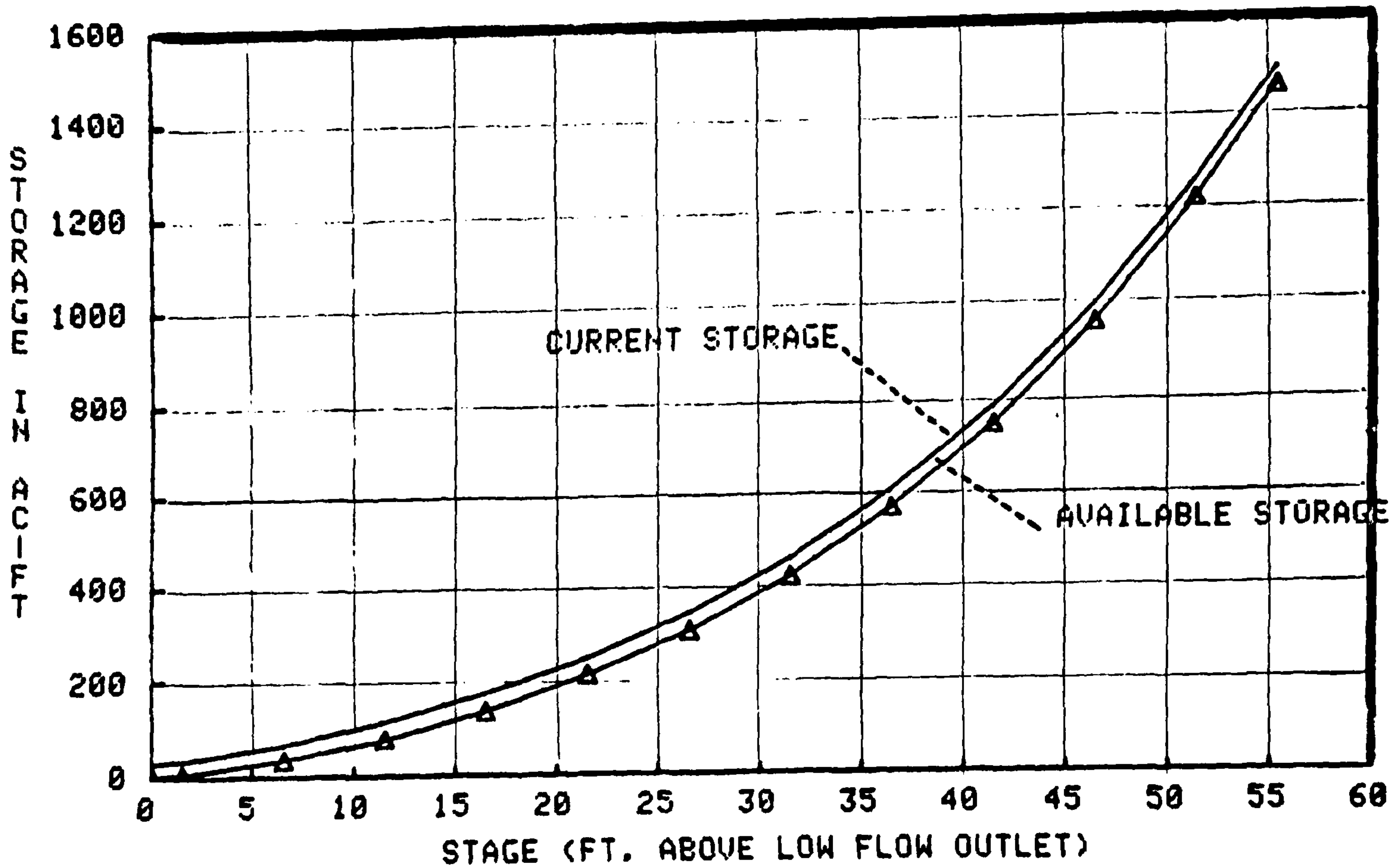
Available storage to Emergency Spillway Crest

$= (1112\text{ ac-ft})(.00381) = 4.24\text{'' of runoff}$

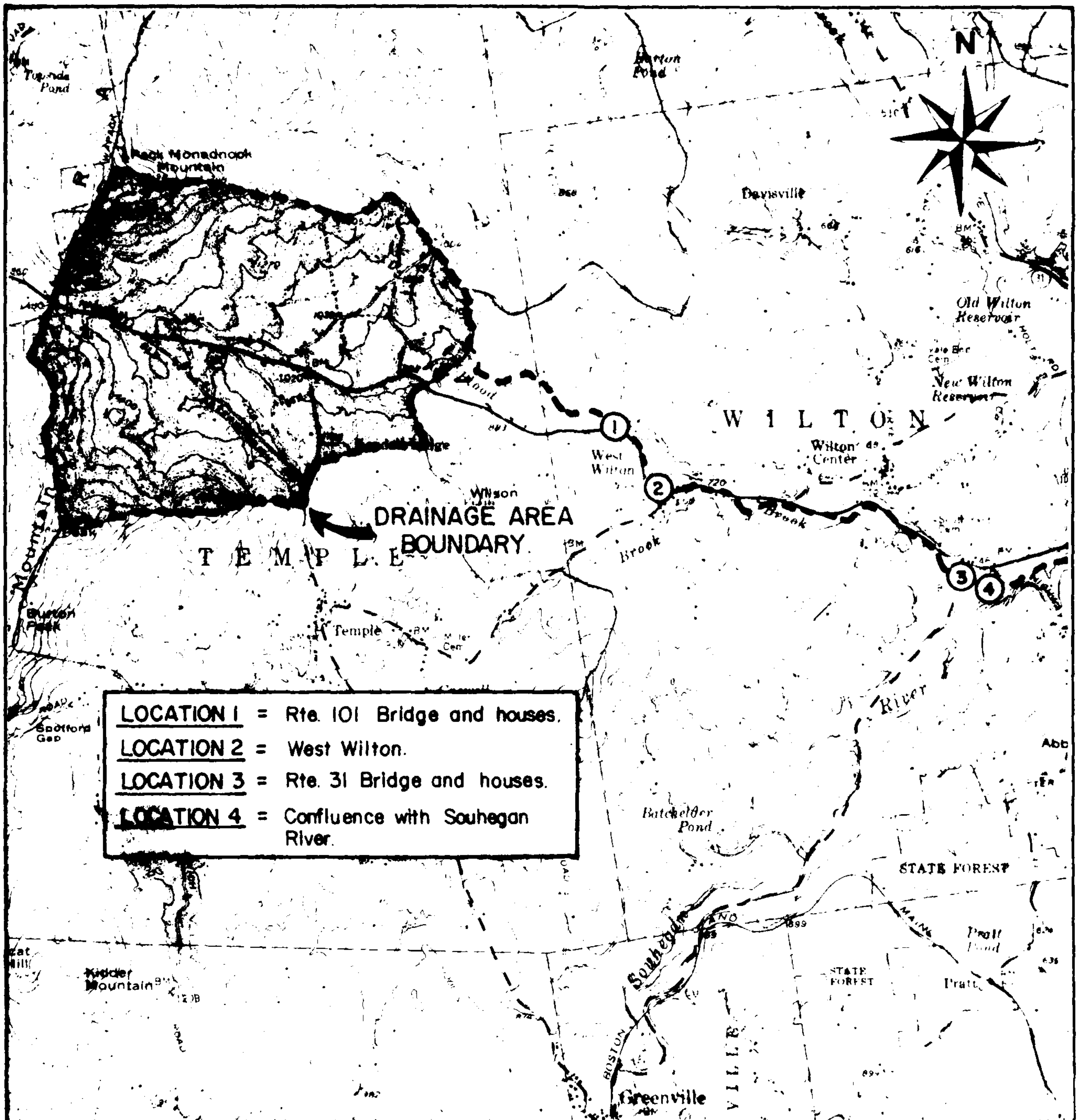
Available Storage to dam crest

$= (1449\text{ ac-ft})(.00381) = 5.52\text{'' of runoff}$

STORAGE-ELEVATION CURVE FOR SOUHEGAN R. W. DAM # 26



P. 11



- SCALE -

0 1/2 1 2 (Miles)

FROM: USGS PETERBOROUGH - N.H. QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCATION AND DOWNSTREAM HAZARD MAP

SOUHEGAN RIVER
WATERSHED DAM No. 26 NEW HAMPSHIRE

FILE No. 2327

SCALE AS NOTED

DATE

Dam Failure Analysis

p. 12 is a location and downstream hazard map for S.R.W.D. # 26.

The first question to be addressed in the Downstream Hazard Analysis is the assumed water surface elevation at dam failure. The normal assumption is that failure occurs with the water surface at the top of the dam. This would yield a pre-failure outflow of 12,694 cfs, which would create severe flooding downstream prior to dam failure. Dam failure would have a greater incremental impact on flooding if it were to occur with a lower water surface elevation in the reservoir. Therefore, for this analysis failure is assumed to occur with the water surface at S.C.S. design high water, 927 ft. MSL, h = 53.5', 2 ft. below the dam crest. This represents 4 ft of flow in the Emergency Spillway, and a pre-failure outflow of 6550 cfs. Available storage at this level is 1332.5 Ac-ft (≅1330 Ac-Ft)

Peak failure = Normal outflow + Breach outflow

Normal Outflow = 6550 cfs

Breach Outflow = $Q_{p_1} = 8/27 \sqrt{g}\, W_b\, y_o^{3/2}$

Where: W_b = breach width = 40% of width at 1/2 height of the dam = .4 (352) = 140.7 ft (D-1 width from Sheet 4 of SCS Plans).

y_o = height above tailwater at time of failure.

Assume tailwater at 860' MSL.

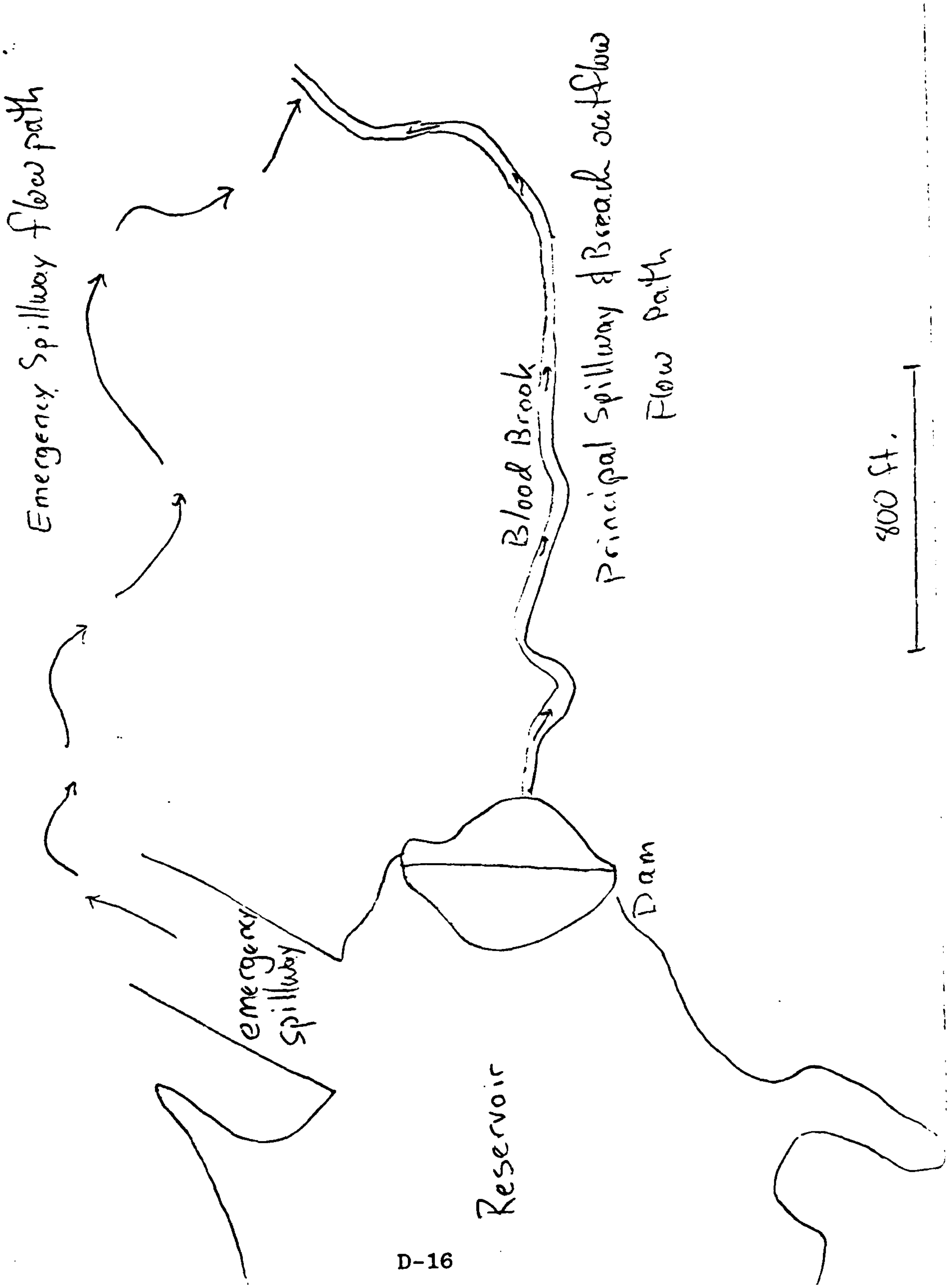
$y_o = 927 - 860 = 67$

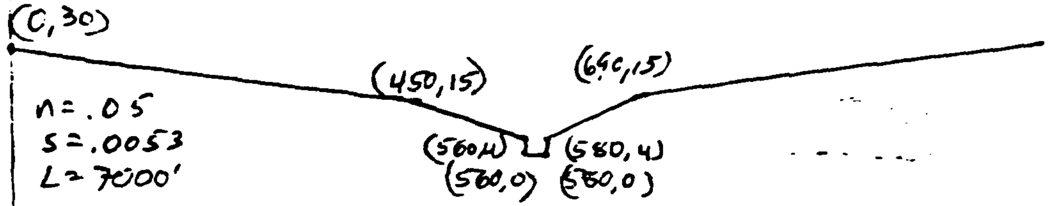
$Q_{p_1} = 8/27 \sqrt{g}\ 140.7\ (67)^{3/2} = 129{,}700$ cfs

Peak failure outflow = 129,700 + 6550 = 136,300 cfs

P. 15 gives a sketch of the flow path of water through the dam breach, the emergency spillway, and the principal spillway. The flows combine about 2000 ft. downstream of the dam.

There is no development near Blood Brook immediately downstream of S.R.W. Dam #26. The first hazard area downstream is Location 1 (see p. 12), 7000' from the dam. The following cross-section for the reach is based on field notes and U.S. G.S. quad sheets. (See p. 16).


Emergency Spillway flow path
Blood Brook
Principal Spillway & Breach outflow
Flow Path
800 ft.
Dam
emergency
spillway
Reservoir



n=.05
s=.0053
L=7000'

A Stage-Normal Discharge relationship for this reach is given on p. 17. At the pre-failure outflow of 6,550 cfs, there would be 12.8' of flow in this reach. The attenuation due to storage in the reach is calculated on pp. 18-19.

The attenuated peak dam failure flow at Highway 101 is 51,700 cfs, which would create a stage of 24 ft. in the Brook at this point.

The only development in this reach consists of two houses about 15 ft. above the streambed. The pre-failure outflow would not cause flooding at these houses, which are about 500 ft. upstream of Highway 101. The peak dam failure flow of 57,100 cfs and the resulting 24.0 stage would create about 9 feet of flooding at these houses. This would certainly cause serious damage and present a threat of loss of life at this location.

The bridge across Blood Brook on Highway 101, a heavily traveled major road, has a low chord about 13 feet above the streambed. Therefore, 24 feet of flow in the natural stream channel would severely overtop and probably destroy this bridge. Also, a bridge on a minor dirt road at the same location as the houses would be destroyed.

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	20.0	22.0	0.9	18.8	40.7
2.00	2.0	40.0	24.0	1.7	56.2	122.0
3.00	3.0	60.0	26.0	2.3	104.8	227.4
4.00	4.0	80.0	28.0	2.9	161.1	349.6
5.00	5.0	110.0	48.1	2.3	191.0	414.3
6.00	6.0	160.0	68.2	2.3	282.6	613.0
7.00	7.0	230.0	88.3	2.6	435.6	944.9
8.00	8.0	320.0	108.4	3.0	658.8	1429.1
9.00	9.0	430.0	128.5	3.3	962.4	2087.9
10.00	10.0	560.0	148.6	3.8	1356.7	2943.4
11.00	11.0	710.0	168.7	4.2	1851.7	4017.2
12.00	12.0	880.0	188.8	4.7	2456.8	5329.9
13.00	13.0	1070.0	208.9	5.1	3181.2	6901.5
14.00	14.0	1280.0	229.0	5.6	4033.8	8751.2
15.00	15.0	1510.0	249.1	6.1	5023.2	10897.6
16.00	16.0	1780.0	309.1	5.8	5721.7	12413.1
17.00	17.0	2110.0	369.2	5.7	6749.1	14641.9
18.00	18.0	2500.0	429.2	5.8	8098.1	17568.6
19.00	19.0	2950.0	489.2	6.0	9778.8	21214.9
20.00	20.0	3460.0	549.3	6.3	11808.8	25618.9
21.00	21.0	4030.0	609.3	6.6	14209.0	30826.0
22.00	22.0	4660.0	669.3	7.0	17001.9	36885.1
23.00	23.0	5350.0	729.4	7.3	20210.7	43846.6
24.00	24.0	6100.0	789.4	7.7	23858.8	51761.0
25.00	25.0	6910.0	849.4	8.1	27969.3	60678.6
26.00	26.0	7780.0	909.5	8.6	32565.2	70649.2
27.00	27.0	8710.0	969.5	9.0	37669.0	81721.8
28.00	28.0	9700.0	1029.5	9.4	43303.0	93944.7
29.00	29.0	10750.0	1089.6	9.9	49489.2	107365.5
30.00	30.0	11860.0	1149.6	10.3	56249.1	122030.8

P.17

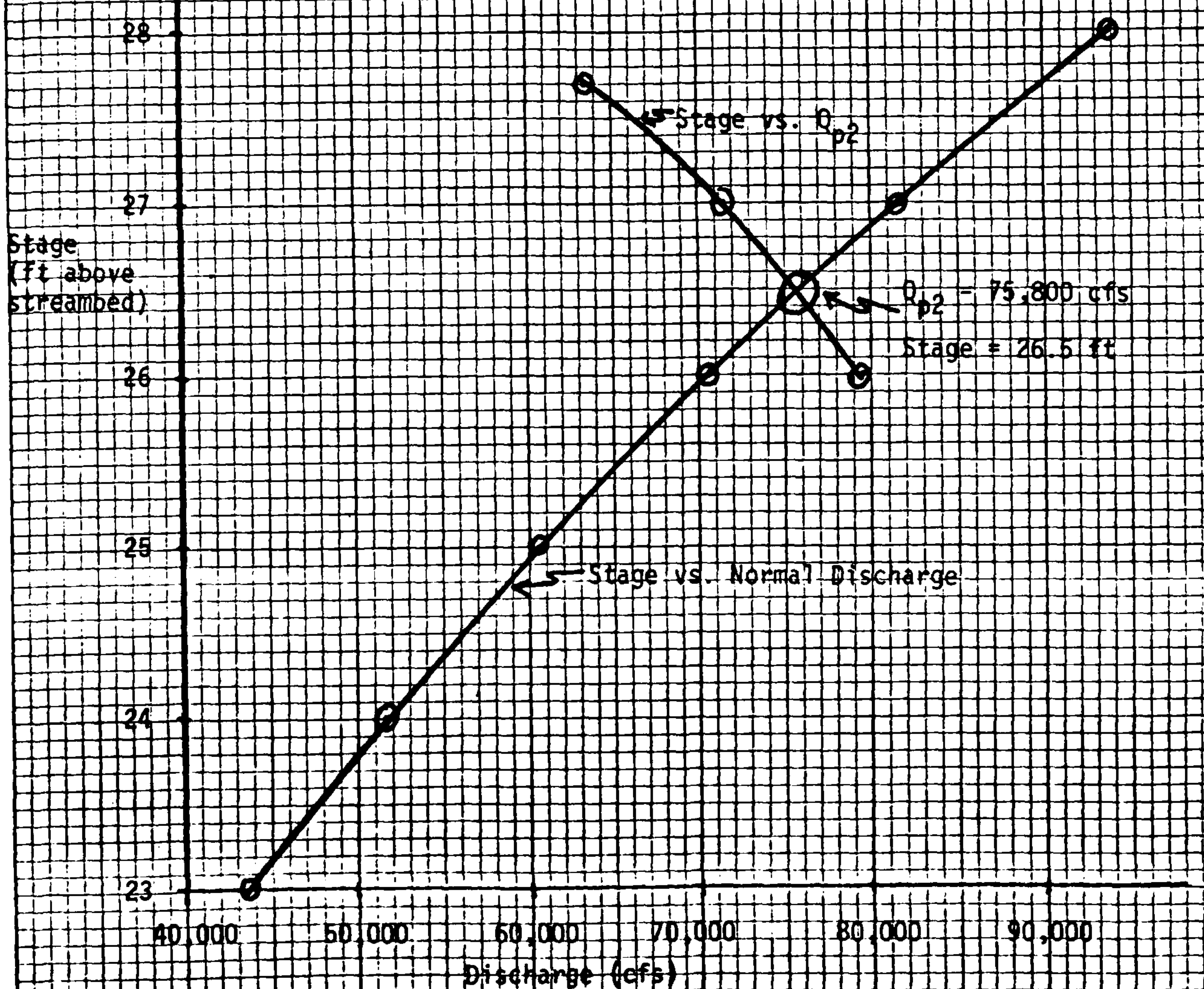
REACH FROM , DOWNSTREAM OF THE DAM TO U.S. HIGHWAY 101

Attenuated Peak Dam Failure Flow, 3500 ft Downstream of Dam

$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 136,300\left(1 - \frac{STOR}{1330}\right)$$

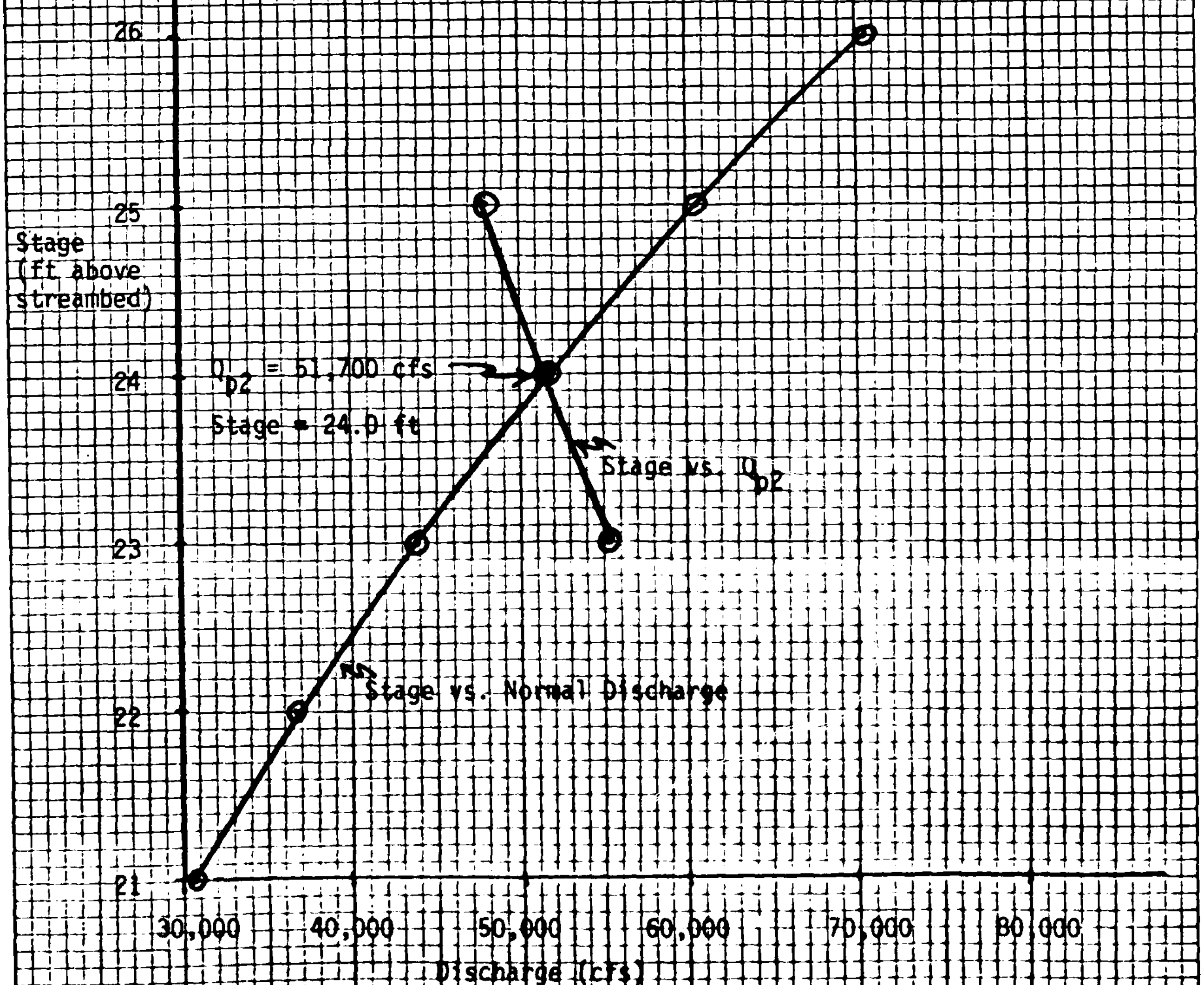
ICG, 6/14/79, p. 18

Stage (ft)	Area (above 12.8 ft) (sq. ft)	Storage $\left(\frac{AREA \times 3500}{43,560}\right)$ (ac ft)	Q_{p2} (cfs)
26	6934	557	79,200
27	7864	632	71,500
28	8854	711	63,400

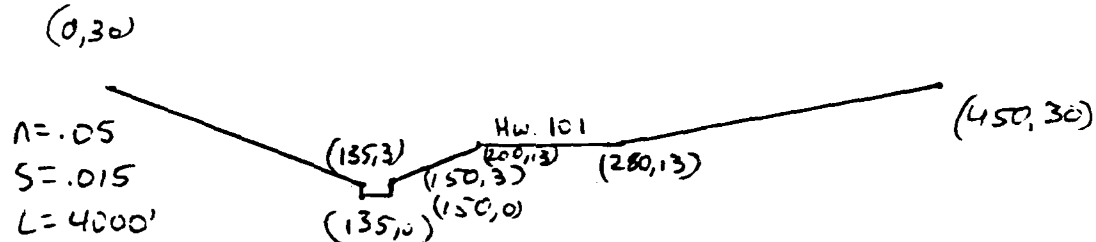


$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 75,800\left(1 - \frac{STOR}{1330}\right)$$

Stage (ft)	Area (above 12.8 ft) (sq ft)	Storage $\left(\frac{AREA \times 3500}{43,560}\right)$ (ac ft)	Q_{p2} (cfs)
23	4504	362	55,200
24	5254	422	51,700
25	6064	487	48,000



The next hazard area downstream of the U.S. Highway 101 Bridge is the village of West Wilton, which is located along Blood Brook. The following typical cross-section for the 4000' reach from the Hw. 101 bridge to Temple Brook, which flows into Blood Brook in the middle of Wilton, is established from fieldnotes and U.S.G.S. topo quads.



A Stage-Normal Discharge relationship for this reach is given on p. 21. At the pre-failure flow of 6550 cfs, there would be 12.1 ft. of flow in the channel. The attenuation due to storage in the reach is calculated on p. 22.

The attenuated peak dam failure flow at the confluence of Temple and Blood Brooks is 47,300 cfs, which would create a stage of about 21.9 ft. in the brook at this point.

The development along Blood Brook in this reach includes two small dams, a bridge across a dirt road, and a bridge on a major road with two concrete arch culverts, 11'X12' and 13'X15' in size. There are also a number of houses in this reach: 2 8'-10' above the

P. 21

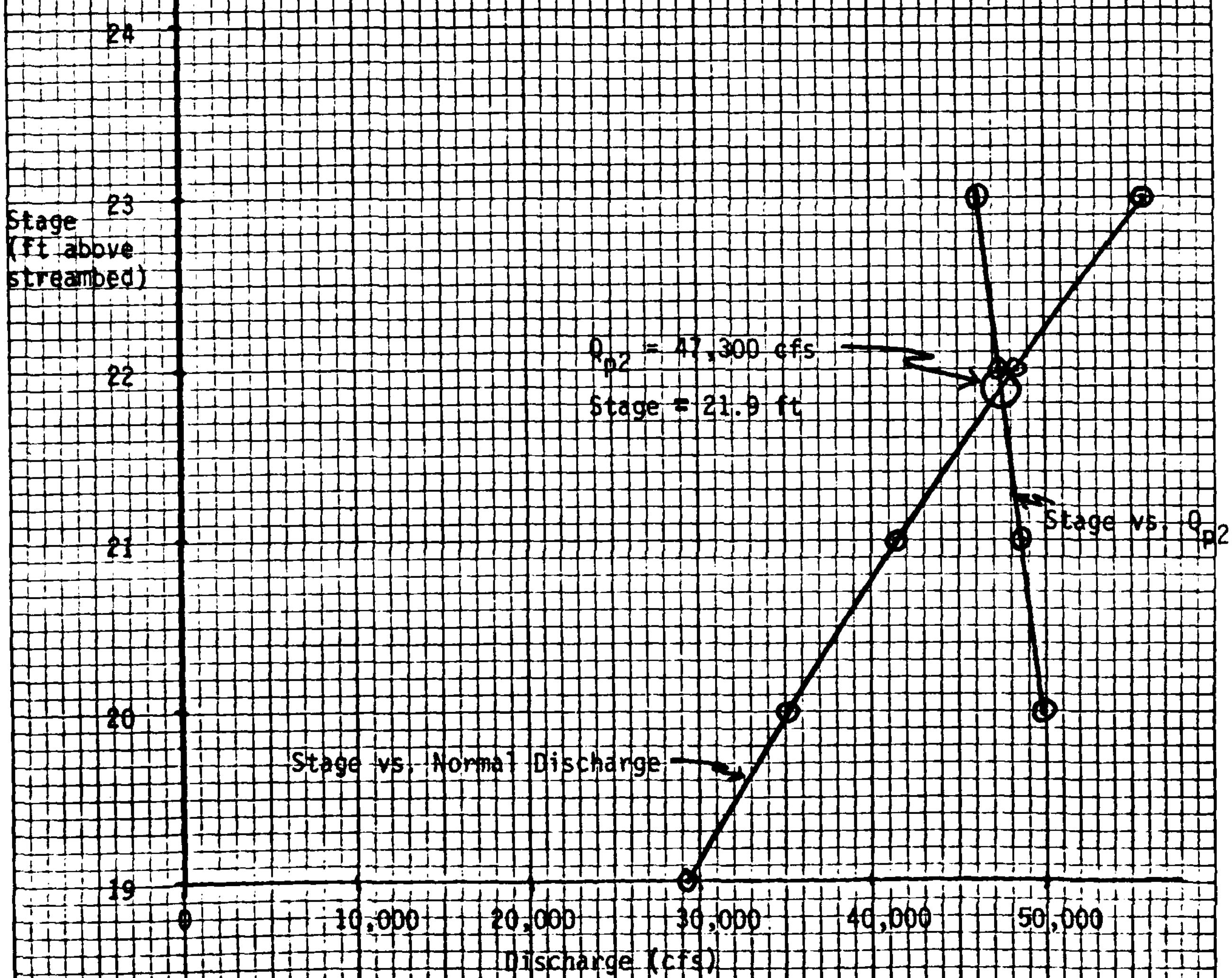
DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	15.0	17.0	0.9	13.8	50.4
2.00	2.0	30.0	19.0	1.6	40.7	148.5
3.00	3.0	45.0	21.0	2.1	74.8	273.1
4.00	4.0	65.0	31.2	2.1	106.1	387.1
5.00	5.0	95.0	41.4	2.3	165.3	603.4
6.00	6.0	135.0	51.6	2.6	256.4	935.9
7.00	7.0	185.0	61.8	3.0	384.4	1403.1
8.00	8.0	245.0	72.0	3.4	554.5	2024.0
9.00	9.0	315.0	82.2	3.8	771.8	2816.9
10.00	10.0	395.0	92.4	4.3	1041.0	3799.5
11.00	11.0	485.0	102.6	4.7	1366.9	4988.9
12.00	12.0	585.0	112.8	5.2	1753.9	6401.3
13.00	13.0	695.0	123.0	5.7	2206.3	8052.5
14.00	14.0	897.5	218.1	4.1	2305.6	8414.9
15.00	15.0	1115.0	233.3	4.8	3165.4	11553.1
16.00	16.0	1347.5	248.4	5.4	4162.2	15191.1
17.00	17.0	1595.0	263.6	6.1	5299.8	19342.8
18.00	18.0	1857.5	278.7	6.7	6582.2	24023.4
19.00	19.0	2135.0	293.9	7.3	8013.9	29248.6
20.00	20.0	2427.5	309.0	7.9	9599.2	35034.7
21.00	21.0	2735.0	324.2	8.4	11342.8	41398.3
22.00	22.0	3057.5	339.3	9.0	13249.2	48356.1
23.00	23.0	3395.0	354.5	9.6	15322.9	55924.6
24.00	24.0	3747.5	369.6	10.1	17568.5	64120.6
25.00	25.0	4115.0	384.8	10.7	19990.7	72960.8
26.00	26.0	4497.5	399.9	11.2	22593.8	82461.6
27.00	27.0	4895.0	415.1	11.8	25382.5	92639.5
28.00	28.0	5307.5	430.2	12.3	28361.2	103510.9
29.00	29.0	5735.0	445.4	12.9	31534.3	115092.0
30.00	30.0	6177.5	460.5	13.4	34906.3	127398.8

REACH FROM 101 BRIDGE TO CONFLUENCE WITH TEMPLE BROOK

Attenuated Peak Dam Failure Flow at Confluence with Temple Brook TCG, 6/14/79, p. 22

$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 57,100\left(1 - \frac{STOR}{1330}\right)$$

Stage (ft)	Area (above 12.1 ft) (sq ft)	Storage $\left(\frac{AREA \times 4000}{43,560}\right)$ (ac ft)	Q_{p2} (cfs)
20	1832	168	49,900
21	2139	196	48,700
22	2462	226	47,400
23	2799	257	46,100

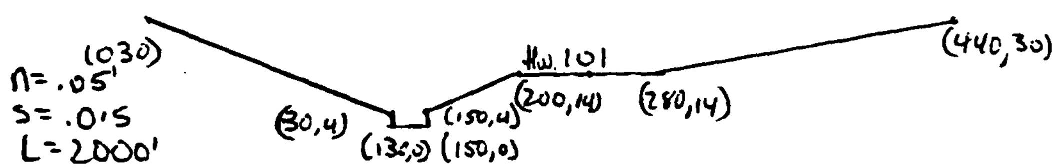


streambed; 3 10'-15' above the streambed; and 3 15'-20' above the streambed.

The pre-failure flow of 6,550 cfs would create fairly serious flooding (2'-4') in the two lowest houses, and minor (<2') in the 3 10'-15' up. The two dams and the small bridge would probably be severely damaged or destroyed.

The peak dam failure flow of 47,300 cfs would create devastating flooding (7'-14') at the 5 lowest houses and 2-7 ft of flooding at the 3 highest houses. The two dams and two bridges would probably be destroyed. There would be high potential for loss of life in this area. Business Highway 101, which parallels Blood Brook in this area, would also be severely flooded [illegible]

Blood Brook continues through the village of West Wilton after it is joined by Temple Brook. The following typical cross-section for the 2000 ft. reach to the end of West Wilton was established from field notes and U.S.G.S. topo information.



A Stage-Normal Discharge relationship for this reach is given on p. 24. At the pre-failure flow of 7,500 cfs (assuming 950 cfs inflow from Temple Brook), there would be 12.9 ft of flow in the channel. The attenuation due to storage in the reach is calculated on p. 25

P.24

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	20.0	22.0	0.9	18.8	68.5
2.00	2.0	40.0	24.0	1.7	56.2	205.3
3.00	3.0	60.0	26.0	2.3	104.8	382.5
4.00	4.0	80.0	28.0	2.9	161.1	588.1
5.00	5.0	105.0	38.2	2.7	206.1	752.3
6.00	6.0	140.0	48.4	2.9	284.3	1037.7
7.00	7.0	185.0	58.6	3.2	398.3	1453.7
8.00	8.0	240.0	68.8	3.5	552.3	2015.7
9.00	9.0	305.0	79.0	3.9	751.0	2741.0
10.00	10.0	380.0	89.2	4.3	999.2	3646.8
11.00	11.0	465.0	99.4	4.7	1301.4	4750.0
12.00	12.0	560.0	109.6	5.1	1662.3	6067.1
13.00	13.0	665.0	119.8	5.6	2086.2	7614.1
14.00	14.0	780.0	130.0	6.0	2577.3	9406.5
15.00	15.0	987.5	225.1	4.4	2647.4	9662.4
16.00	16.0	1210.0	240.3	5.0	3556.9	12981.6
17.00	17.0	1447.5	255.4	5.7	4603.7	16802.2
18.00	18.0	1700.0	270.6	6.3	5791.9	21138.9
19.00	19.0	1967.5	285.7	6.9	7125.9	26007.6
20.00	20.0	2250.0	300.9	7.5	8610.1	31424.5
21.00	21.0	2547.5	316.0	8.1	10249.0	37406.1
22.00	22.0	2860.0	331.2	8.6	12047.2	43969.2
23.00	23.0	3187.5	346.3	9.2	14009.4	51130.7
24.00	24.0	3530.0	361.5	9.8	16140.1	58907.2
25.00	25.0	3887.5	376.6	10.3	18444.0	67315.7
26.00	26.0	4260.0	391.8	10.9	20925.5	76372.7
27.00	27.0	4647.5	406.9	11.4	23589.3	86094.8
28.00	28.0	5050.0	422.1	12.0	26439.8	96498.5
29.00	29.0	5467.5	437.2	12.5	29481.6	107600.2
30.00	30.0	5900.0	452.4	13.0	32719.0	119416.0

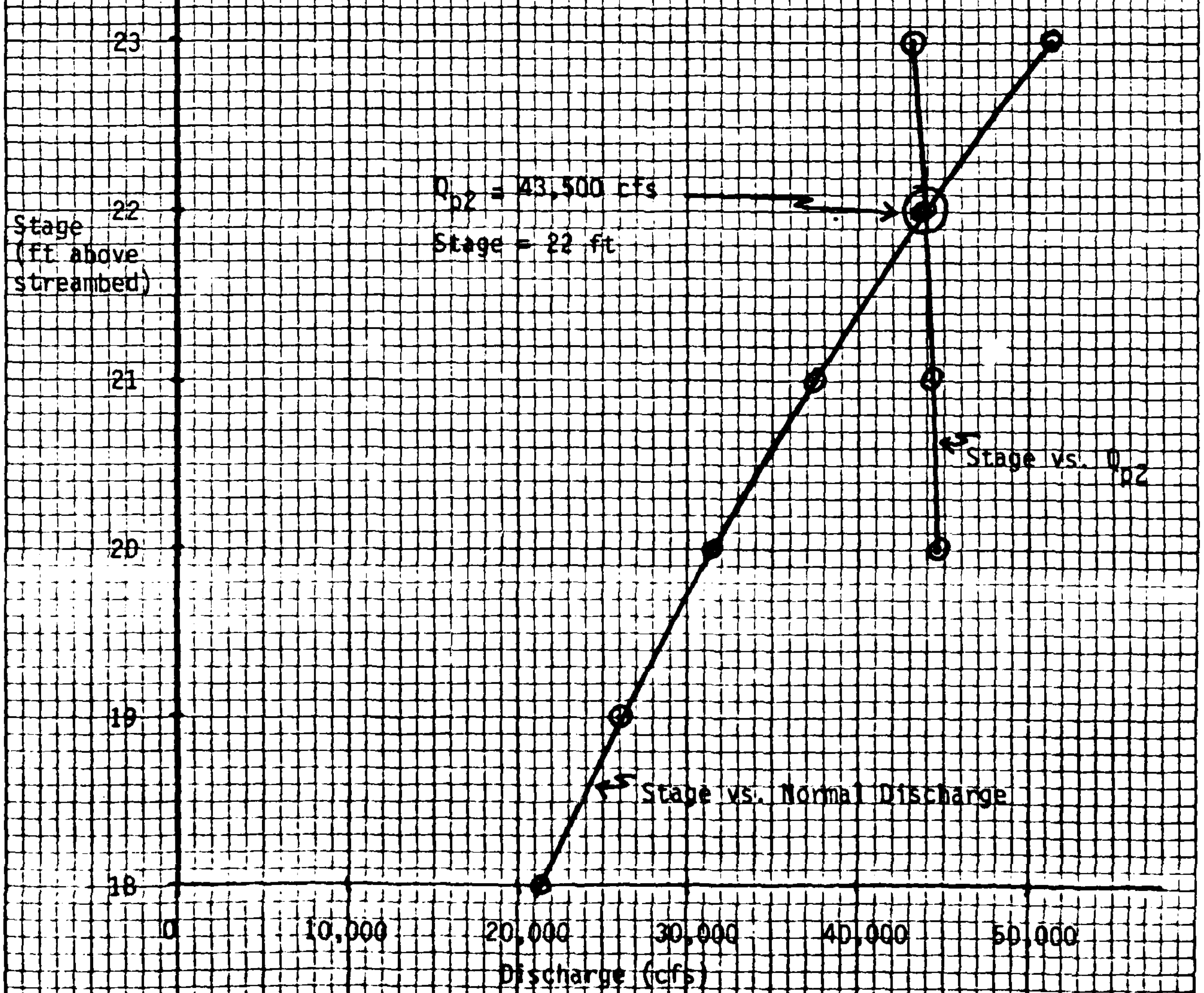
REACH FROM CONFLUENCE WITH TEMPLE BROOK TO END OF WEST WILTON

Attenuated Peak Dam Failure Flow at Downstream End of West Wilton

TCG, 6/14/79, p. 25

$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 47,300\left(1 - \frac{STOR}{1330}\right)$$

Stage (ft)	Area (above 12.9 ft) (sq ft)	Storage $\left(\frac{AREA \times 2000}{43,560}\right)$ (ac ft)	Q_{p2} (cfs)
20	1596	73.3	44,700
21	1893	86.9	44,200
22	2206	101	43,700
23	2533	116	43,100

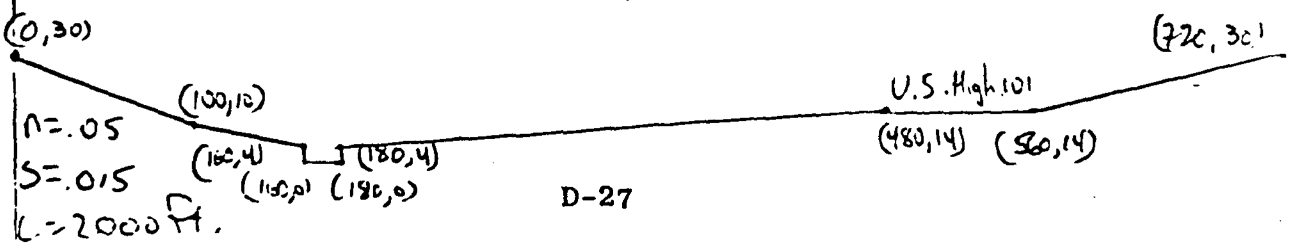


The attenuated peak dam failure flow at the downstream end of West Wilton is 43,500 cfs, which would create a stage of 22 feet in the brook at this point.

The development along Blood Brook in this reach includes one group of 3 houses 10'-15' above the streambed, and a gift shop and restaurant 12' above the streambed. The pre-failure flow of 7500 cfs would create slight flooding at the restaurant and gift shop, and slight (22.9 ft) flooding at the houses.

The peak dam failure flow of 43,500 cfs would create 910 ft. of flooding at the gift shop and restaurant, and 7-12 ft. of flooding at the 3 houses. There would be high potential for loss of life in this area. Also, Business Highway 101, which parallels Blood Brook in this area, would be severely flooded and damaged.

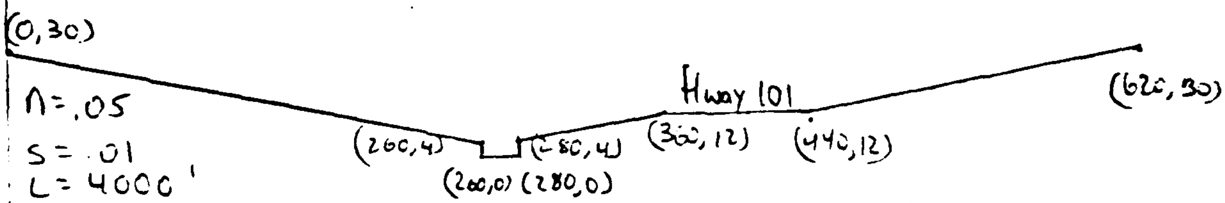
The next damage center downstream of West Wilton is a house 2000' downstream on Blood Brook. The following typical cross-section for the reach was established from field notes and U.S.G.S. topo information.



A Stage-Normal Discharge relationship for this reach is given on p. 28. At the pre-failure flow of 7,500 cfs, there would be 9.9 ft. of flow in the channel. The attenuation due to storage in this reach is calculated on p.29.

The peak dam failure flow of 40,000 cfs would create a stage of 16 ft. after dam failure. The house at the end of the reach is 10'-15' above the streambed, so dam failure would increase flooding from none to 1-6 ft. Also, Highway 101, which parallels Blood Brook in this area, would be slightly flooded.

The next hazard area downstream is a house at an abandoned mill and mill pond 4000' along Blood Brook. The following typical cross-section for the reach was established from field notes and U.S.G.S. topo information.



A Stage-Normal Discharge relationship for this reach is given on p. 30. At the pre-failure flow of 7,500 cfs, the would be 12.1 ft. of flow in the channel. The attenuation due to storage in this reach is calculated on p.31.

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	20.0	22.0	0.9	18.8	68.5
2.00	2.0	40.0	24.0	1.7	56.2	205.3
3.00	3.0	60.0	26.0	2.3	104.8	382.5
4.00	4.0	80.0	28.0	2.9	161.1	588.1
5.00	5.0	120.0	68.1	1.8	175.2	639.3
6.00	6.0	200.0	108.1	1.8	301.4	1100.1
7.00	7.0	320.0	148.2	2.2	534.7	1951.6
8.00	8.0	480.0	188.3	2.5	896.1	3270.5
9.00	9.0	680.0	228.3	3.0	1408.1	5139.1
10.00	10.0	920.0	268.4	3.4	2092.4	7636.6
11.00	11.0	1197.5	303.5	3.9	2991.4	10917.7
12.00	12.0	1510.0	338.6	4.5	4092.9	14937.9
13.00	13.0	1857.5	373.7	5.0	5412.5	19754.1
14.00	14.0	2240.0	408.[illegible]	5.5	6965.3	25421.6
15.00	15.0	2727.5	504.0	5.4	8411.9	30701.1
16.00	16.0	3230.0	519.2	6.2	10932.9	39902.2
17.00	17.0	3747.5	534.3	7.0	13740.1	50147.8
18.00	18.0	4280.0	549.5	7.8	16829.9	61424.6
19.00	19.0	4827.5	564.6	8.6	20200.0	73724.6
20.00	20.0	5390.0	579.8	9.3	23849.2	87043.5
21.00	21.0	5967.5	594.9	10.0	27777.3	101379.9
22.00	22.0	6560.0	610.1	10.8	31984.4	116734.8
23.00	23.0	7167.5	625.2	11.5	36471.4	133111.1
24.00	24.0	7790.0	640.4	12.2	41239.4	150513.0
25.00	25.0	8427.5	655.5	12.9	46289.9	168946.0
26.00	26.0	9080.0	670.6	13.5	51624.6	188416.4
27.00	27.0	9747.5	685.8	14.2	57245.6	208931.6
28.00	28.0	10430.0	700.9	14.9	63155.1	230499.5
29.00	29.0	11127.5	716.1	15.5	69355.2	253128.5
30.00	30.0	11840.0	731.2	16.2	75848.6	276827.7

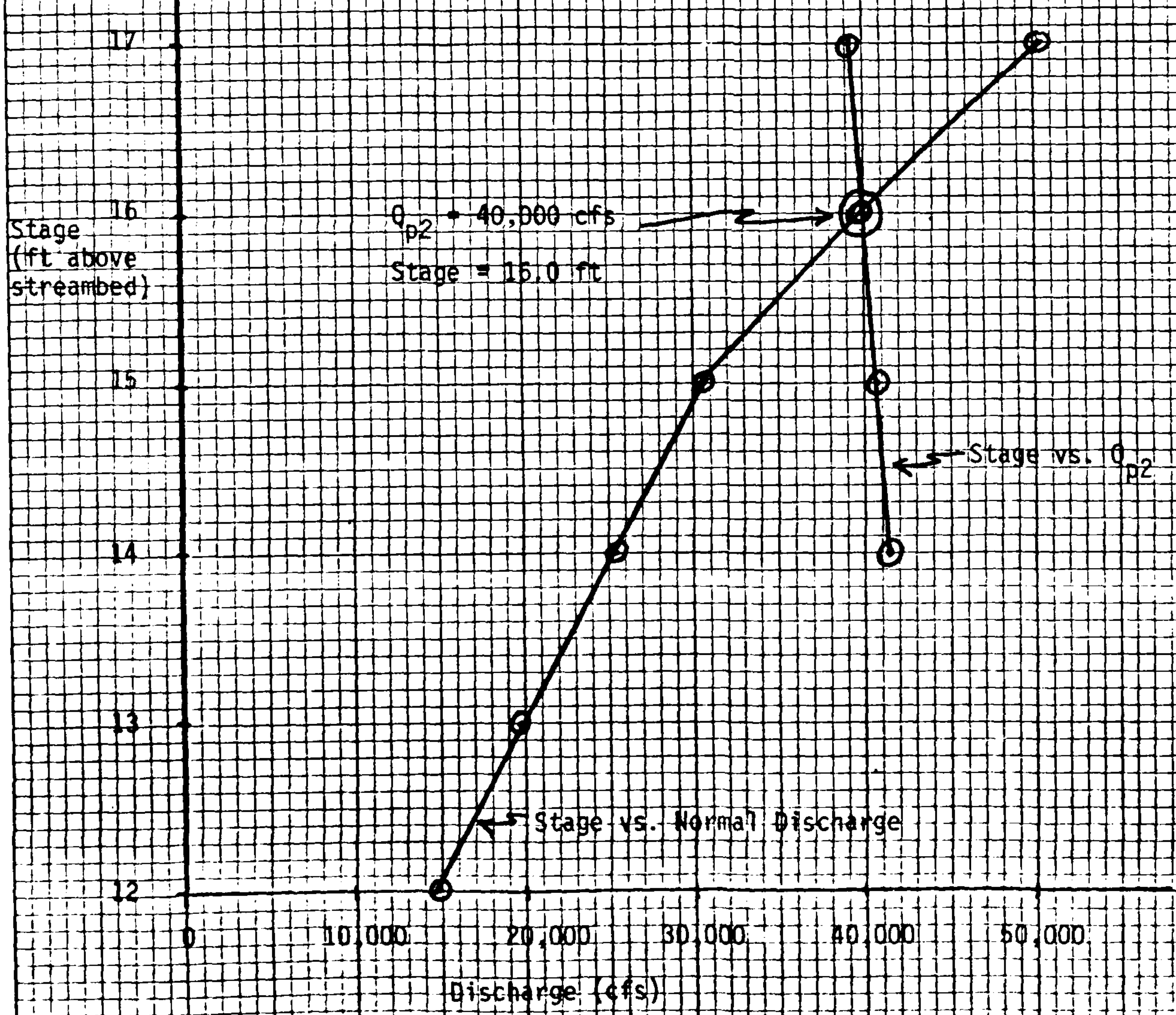
p.28

REACH FROM WEST WILTON TO HOUSE 2000 FT. DOWNSTREAM

Attenuated Peak Dam Failure Flow at House 2000 ft Downstream of West Wilton

$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 43{,}500\left(1 - \frac{STOR}{1330}\right)$ TCG, 5/14/79, p. 29

Stage (ft)	Area (above 9.9 ft) (sq ft)	Storage $\left(\frac{AREA \times 2000}{43{,}560}\right)$ (ac ft)	Q_{p2} (cfs)
14	1344	61.7	41,500
15	1832	84.1	40,800
16	2334	107.	40,000
17	2850	131.	39,200



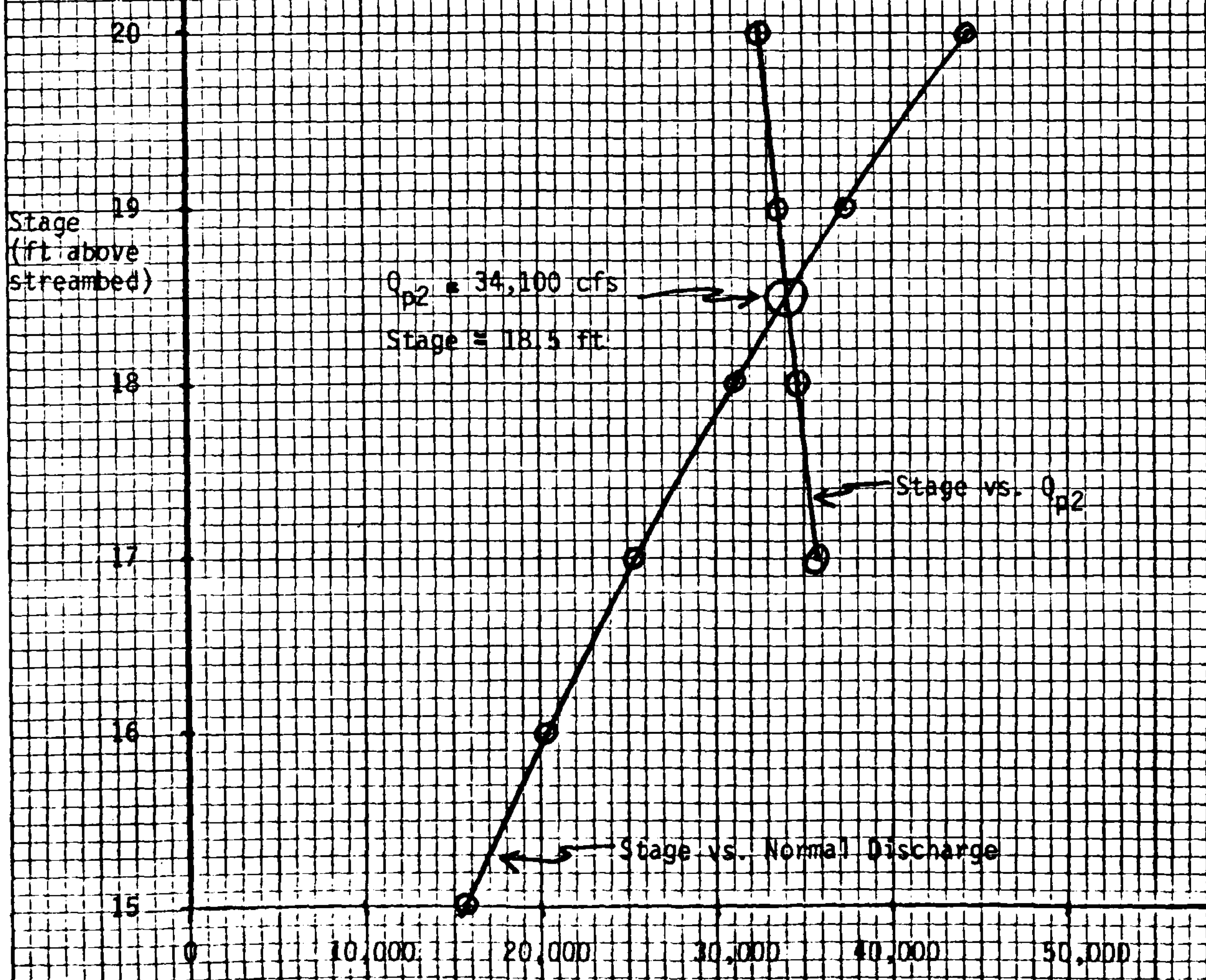
P.30

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	20.0	22.0	0.9	18.8	55.9
2.00	2.0	40.0	24.0	1.7	56.2	167.6
3.00	3.0	60.0	26.0	2.3	104.8	312.3
4.00	4.0	80.0	28.0	2.9	161.1	480.2
5.00	5.0	110.0	48.1	2.3	191.0	569.2
6.00	6.0	160.0	68.2	2.3	282.6	842.1
7.00	7.0	230.0	88.3	2.6	435.6	1298.0
8.00	8.0	320.0	108.4	3.0	658.8	1963.1
9.00	9.0	430.0	128.5	3.3	962.4	2868.0
10.00	10.0	560.0	148.6	3.8	1356.7	4043.1
11.00	11.0	710.0	168.7	4.2	1851.7	5518.0
12.00	12.0	880.0	188.8	4.7	2456.8	7321.1
13.00	13.0	1150.0	288.9	4.0	2889.8	8611.6
14.00	14.0	1440.0	309.0	4.7	4019.7	11978.6
15.00	15.0	1750.0	329.1	5.3	5334.4	15896.6
16.00	16.0	2080.0	349.2	6.0	6838.9	20379.8
17.00	17.0	2430.0	369.3	6.6	8538.2	25443.8
18.00	18.0	2800.0	389.4	7.2	10438.1	31105.5
19.00	19.0	3190.0	409.5	7.8	12544.4	37382.2
20.00	20.0	3600.0	429.6	8.4	14863.0	44291.6
21.00	21.0	4030.0	449.7	9.0	17399.9	51851.7
22.00	22.0	4480.0	469.8	9.5	20161.2	60080.5
23.00	23.0	4950.0	489.9	10.1	23153.0	68996.1
24.00	24.0	5440.0	510.0	10.7	26381.3	78616.4
25.00	25.0	5950.0	530.1	11.2	29852.2	88959.5
26.00	26.0	6480.0	550.2	11.8	33571.5	100043.2
27.00	27.0	7030.0	570.3	12.3	37545.4	111885.2
28.00	28.0	7600.0	590.4	12.9	41779.6	124503.3
29.00	29.0	8190.0	610.5	13.4	46280.2	137914.9
30.00	30.0	8800.0	630.6	14.0	51052.8	152137.4

REACH FROM HOUSE D/S OF WEST WILTON TO ABANDONED MILL

$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1330}\right) = 40{,}000\left(1 - \frac{STOR}{1330}\right)$$

Stage (ft)	Area (above 12.1 ft) (sq ft)	Storage $\left(\frac{AREA \times 4000}{43{,}560}\right)$ (ac ft)	Q_{p2} (cfs)
17	1523	140	35,800
18	1893	174	34,800
19	2283	210	33,700
20	2693	247	32,600

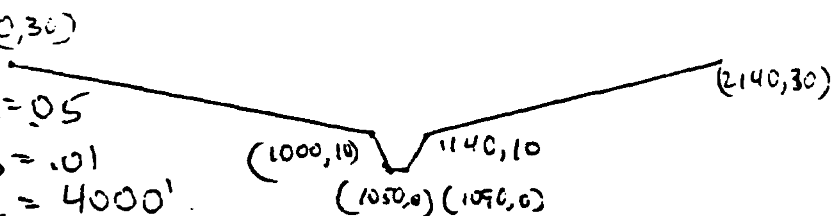


The attenuated peak dam failure flow at the abandoned mill and mill pond is 34,100 cfs, which would create a stage of 18.5 ft. in Blood Brook.

The house near the mill pond is 15 ft. above the streambed, on the far side of Highway 101. The pre-failure flow of 7500 cfs would create a stage of 12.1 ft, about at Highway level.

The peak dam failure flow of 34,100 cfs would create a stage of 18.5 ft, causing 3 ft. of flooding at the house, and about 6 ft. on Highway 101.

The next damage center is at the Highway 31 bridge across Blood Brook. (The sign on the bridge says Gambol Brook). The following typical cross-section for the 4000 ft. reach to the bridge was established from field notes and U.S.G.S. topo data.



n = .05
S = .01
L = 4000'

A Stage-Normal Discharge relationship for this reach is given on p.33. At the pre-failure flow of 7,500 cfs, there would be 9.1 ft. of flow in the channel. The attenuation due to storage in this reach is calculated on

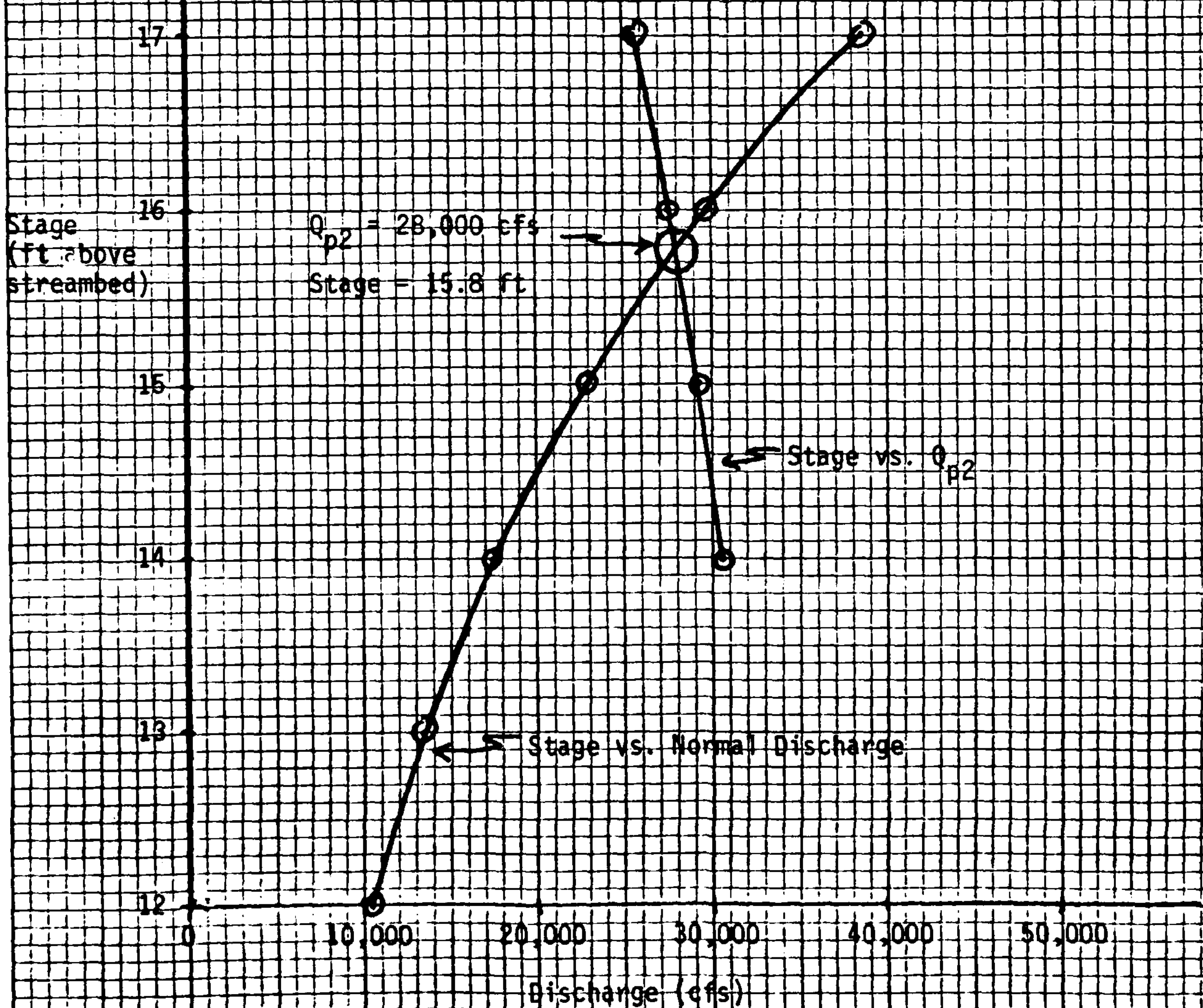
P.33

DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.00	0.0	0.0	0.0	0.0	0.0	0.0
1.00	1.0	45.0	50.2	0.9	41.8	124.7
2.00	2.0	100.0	60.4	1.7	140.0	417.1
3.00	3.0	165.0	70.6	2.3	290.7	866.2
4.00	4.0	240.0	80.8	3.0	496.1	1478.5
5.00	5.0	325.0	91.0	3.6	759.7	2264.0
6.00	6.0	420.0	101.2	4.2	1085.3	3234.1
7.00	7.0	525.0	111.4	4.7	1476.6	4400.3
8.00	8.0	640.0	121.6	5.3	1937.7	5774.4
9.00	9.0	765.0	131.8	5.8	2472.4	7367.8
10.00	10.0	900.0	142.0	6.3	3084.5	9191.8
11.00	11.0	1090.0	242.0	4.5	2974.3	8863.4
12.00	12.0	1380.0	342.0	4.0	3499.2	10427.5
13.00	13.0	1770.0	442.0	4.0	4465.3	13306.5
14.00	14.0	2260.0	542.1	4.2	5857.2	17454.4
15.00	15.0	2850.0	642.1	4.4	7701.3	22949.9
16.00	16.0	3540.0	742.1	4.8	10036.7	29909.3
17.00	17.0	4330.0	842.1	5.1	12906.3	38460.7
18.00	18.0	5220.0	942.1	5.5	16353.9	48734.7
19.00	19.0	6210.0	1042.2	6.0	20423.1	60860.8
20.00	20.0	7300.0	1142.2	6.4	25156.5	74966.4
21.00	21.0	8490.0	1242.2	6.8	30595.9	91175.8
22.00	22.0	9780.0	1342.2	7.3	36782.1	109610.6
23.00	23.0	11170.0	1442.2	7.7	43754.9	130389.6
24.00	24.0	12660.0	1542.3	8.2	51553.2	153628.6
25.00	25.0	14250.0	1642.3	8.7	60215.2	179441.3
26.00	26.0	15940.0	1742.3	9.1	69778.1	207938.7
27.00	27.0	17730.0	1842.3	9.6	80278.4	239229.7
28.00	28.0	19620.0	1942.3	10.1	91752.1	273421.4
29.00	29.0	21610.0	2042.4	10.6	104234.4	310618.6
30.00	30.0	23700.0	2142.4	11.1	117759.9	350924.5

REACH FROM ABANDONED MILL TO HIGHWAY 31 BRIDGE

$$Q_{p2} = Q_{p1}\left(1 - \frac{STOR}{1830}\right) = 34,100\left(1 - \frac{STOR}{1830}\right)$$

Stage (ft)	Area (above 9.1 ft) (sq ft)	Storage $\left(\frac{AREA \times 4000}{43,560}\right)$ (ac ft)	Q_{p2} (cfs)
14	1482	136	30,600
15	2072	190	29,200
16	2762	254	27,600
17	3552	376	25,700



The attenuated peak failure flow at the Highway 31 Bridge would be 28,000 cfs, which would create a stage of 15.8 ft. in Blood Brook.

The Highway 31 Bridge has a low chord about 15' above the stream, and would probably not be seriously damaged by this flow. A junkyard about 15 ft. above the streambed and 3 houses 20 ft. up are probably also above serious flooding.

About 800 ft. downstream of the Highway 31 bridge, Blood Brook enters the Souhegan River. The peak dam failure flow of about 28000 cfs would be attenuated rapidly in the larger river. However, there is one area where, depending on antecedent flows in the Souhegan, the dam failure outflow from S.R.W. Dam #26 could cause serious flooding. About 4000 ft. downstream of the confluence of Blood Brook & the Souhegan River, there is a group of 30-40 houses about 15 ft. above the bed of the river. These houses might experience flooding due to failure outflows from S.R.W. Dam #26. About 4 miles downstream of its confluence with Blood Brook, the Souhegan enters Wilton.

The downstream impacts of the failure of S.R.W. Dam #26 are summarized on p. 36.

183 Dam Safety Souhegan R. W. Dam #26 TCG, 6/14/7?, p.3?

LOCATION # (MAP, P. 12)	Location	# of dwellings	level above streambed (ft)	Flow & Stage before failure	Flow & Stage after failure	Comments
—	at dam	—	—	6550 cfs	136,300 cfs	
1	houses, Rt. 101 Bridge	2	15 ±	6550 cfs 12.8 ft.	57,100 cfs 24 ft.	Rt. 101 bridge severely overtopped
2	West Wilton, u/s of Temple Bk.	2 3 3	8-10 10-15 15-20	6550 cfs 12.1 ft.	47,300 cfs 21.9 ft.	danger of loss of life. 2 dams, 2 bridges, U.S. Rt. 101 also flooded.
2	West Wilton d/s of Temple Brook	1 restaurant 1 gift shop 3 houses	12 12 10-15	7500 cfs 12.9 ft.	43,500 cfs 22 ft.	danger of loss of life. Also floods Rt. 101.
	House, 2000 ft. d/s of West Wilton	1	10-15	7500 cfs 9.9 ft.	40,000 cfs 16 ft.	Rt. 101 flooded
	House @ abandoned mill	1	15	7,500 12.1 ft.	34,100 cfs 18.5 ft.	Rt. 101, old mill & mill dam flooded
3	Highway 31 Bridge	1 junkyard 3 houses	15 20	7500 cfs 9.1 ft.	28,000 cfs 15.8 ft.	probably no damage
4	Souhegan confluence	—	—	7500 cfs	28,000 cfs	—
	d/s on Souhegan	30-40	15 ±	—	—	4000' d/s might be flooded
	Wilton	10-15 near river	varies	—	—	probably attenuated by now.

Test Flood Analysis

Size Classification. Intermediate

Hazard Classification: High

The hazard classification is HIGH due to the potential for serious economic losses and severe loss of life at numerous locations downstream of the dam in the event of dam failure. (See chart, p. 36)

Test Flood: PMF

Using the COE NED "Maximum Probable Flood Peak Flow Rates," the upstream drainage area of 4.9 sq. mi. with mountainous terrain would yield a PMF peak inflow of 2160 csm

Peak inflow = 4.9 (2160) = 10,600 cfs

The SCS "Freeboard Hydrograph" for this dam, which is approximately equivalent to the PMF, has a peak inflow of 13,760 cfs (p. 32, SCS "Hydrology & Hydraulics" Design Calcs.) Since the SCS Freeboard Hydrograph inflow is larger (& therefore more conservative), we will use that as the Test Flood. The S.C.S. used a storage routing procedure to establish their peak

outflow of 11,900 cfs, which would yield a peak Test Flood Elevation of 928.8 ft. MSL, 55.3 ft above normal pool, and 0.2 ft. below the top of the dam. (Note: The SCS starts their routing with the water surface at 892 ft MSL, 18.5 ft above normal pool, the 5-day drawdown elevation.)

DRAWDOWN TIME.

ELEVATION (Ft. MSL)	INTERVAL STORAGE (AC-FT)	Discharge (See pp. 6-7) (cfs)	Average Discharge (CFS)	Discharge (AC-FT/DAY) (1.9835 x col 4)	Drawdown Time (Days) (col 2 ÷ col 5)	Accumulated Time (Days)
E.S.-CREST 923		144				
	161.7		142.5	282.65	.57	
920		141				.57
	216		138	273.7	.79	
915		135				1.36
	178.3		132	261.8	.68	
910		129				2.04
	99.4		127	251.9	.39	
907		125				2.43
	46		97.5	198.4	.24	
RISER CREST 905		70				2.67
	73.6		64	126.9	.58	
902		58				3.25
	115		56	111.1	1.04	
896		52				4.29
	64.8		49.5	98.18	.66	
892		47				4.95
	53		44.5	88.3	.60	
888		42				5.55
	75		34.5	68.4	1.10	
880		27				6.65
	29.7		18.25	36.2	.82	
875		9.5				7.47
	5.5		4.75	9.4	.58	
873.5		0				8.05

This calculation follows p. 29 of SCS "Hydrology & Hydraulics" Calculations, with slightly modified Stage-discharge curve.

APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

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